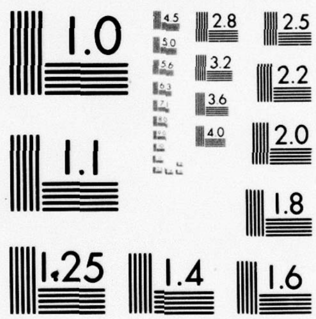
AD-A077 372 ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG--ETC F/G 13/13
CONCRETE AND ROCK TESTS, REHABILITATION WORK, MARSEILLES DAM, I--ETC(U)
SEP 79 R L STOWE

UNCLASSIFIED WES-SL-79-21 NL

1 OF 2

AD A077372



MICROCOPY RESOLUTION TEST CHART

MISCELLANEOUS PAPER SL-79-21

CONCRETE AND ROCK TESTS, REHABILITATION WORK, MARSEILLES DAM, ILLINOIS WATERWAY CHICAGO DISTRICT

by

Richard L. Stowe

Structures Laboratory
U. S. Army Engineer Waterways Experiment Station
P. O. Box 631, Vicksburg, Miss. 39180

September 1979
Final Report

Prepared for U. S. Army Engineer District, Chicago
Chicago, Ill. 60604

14 WES-SL-79-21

Unclassified

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

REPORT DOCUMENTATION PAGE

READ INSTRUCTIONS BEFORE COMPLETING FORM

1. REPORT NUMBER
Miscellaneous Paper SL-79-21

2. GOVT ACCESSION NO.

3. RECIPIENT'S CATALOG NUMBER

4. TITLE (and Subtitle)
6 CONCRETE AND ROCK TESTS, REHABILITATION WORK, MARSEILLES DAM, ILLINOIS WATERWAY, CHICAGO DISTRICT

5. TYPE OF REPORT & ~~PERIOD COVERED~~ 9
Final report. Jan-Mar 77

6. PERFORMING ORG. REPORT NUMBER

7. AUTHOR(s)
10 Richard L. Stowe

8. CONTRACT OR GRANT NUMBER(s)

9. PERFORMING ORGANIZATION NAME AND ADDRESS
U. S. Army Engineer Waterways Experiment Station
Structures Laboratory
P. O. Box 631, Vicksburg, Miss. 39180

10. PROGRAM ELEMENT, PROJECT, TASK AREA & WORK UNIT NUMBERS
12 102

11. CONTROLLING OFFICE NAME AND ADDRESS
U. S. Army Engineer District, Chicago
Chicago, Ill. 60604

12. REPORT DATE
11 Sep~~tember 19~~79

13. NUMBER OF PAGES
75

14. MONITORING AGENCY NAME & ADDRESS (if different from Controlling Office)

15. SECURITY CLASS. (of this report)
Unclassified

15a. DECLASSIFICATION/DOWNGRADING SCHEDULE

16. DISTRIBUTION STATEMENT (of this Report)

Approved for public release; distribution unlimited.

17. DISTRIBUTION STATEMENT (of the abstract entered in Block 20, if different from Report)

18. SUPPLEMENTARY NOTES

19. KEY WORDS (Continue on reverse side if necessary and identify by block number)

Concrete tests	Grouting
Dam foundations	Illinois Waterway
Dam stability	Marseilles Dam
Field tests	Rock tests (Laboratory)

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

Drilling for field testing and laboratory testing was carried out as part of a stabilization program at the Marseilles Dam on the Illinois Waterway. An investigation conducted in early 1973 showed that tainter gate sections failed to meet current overturning criteria requirements. It was recommended that the gate sections be stabilized by installation of grouted, prestressed tendons. Physical property data of concrete and foundation rock for use in a stability analysis and the design of an anchorage system of grouted tendons (Continued)

cont'

DD FORM 1 JAN 73 1473 EDITION OF 1 NOV 65 IS OBSOLETE

Unclassified

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

411 415

JUB

20. ABSTRACT (Continued)

are presented in this report. Field testing consisted of down-hole televiewing for purposes of obtaining the orientation of natural discontinuities in the foundation rock. Pressure transducers were used in boreholes to ascertain uplift pressures adjacent to the boreholes. Laboratory testing included the determination of characterization properties (compressive strength, unit weight, tensile strength, and compressional wave velocities) and engineering design properties (elastic moduli, triaxial strengths, direct shear of intact and discontinuous rock specimens, and rebar pullout resistance). An assessment of the foundation conditions was made and guidance is presented as to proper choice of design values for the foundation rock.

PREFACE

This testing program, "Concrete and Rock Tests, Rehabilitation Work: Marseilles Dam, Illinois Waterway, Chicago District," was conducted for the U. S. Army Engineer District, Chicago. The work was authorized by DA Form 2544, NCC-IA-77-08, dated 17 November 1976.

Drilling was conducted by personnel of the Explorations Branch of the Soils and Pavements Laboratory (S&PL) of the U. S. Army Engineer Waterways Experiment Station (WES) during the period October-December 1976 under the direction of Mr. Mark Vispi. Laboratory tests were performed at the Concrete Laboratory (CL) and the S&PL of the WES during the period January-March 1977 under the direction of Messrs. Bryant Mather and John M. Scanlon, CL. Mr. W. B. Steinriede, S&PL, conducted the televiewer logging; Mr. D. L. Ainsworth, CL, conducted the pressure transducer tests; Mr. G. P. Hale supervised the laboratory testing that was conducted in the S&PL; and Mr. G. S. Wong conducted the petrographic examination. The stability analysis and design of an anchorage system referenced in this report were conducted by Dr. C. E. Pace of the CL. Mr. R. L. Stowe was Project Leader and was assisted in performing laboratory work at the CL by Messrs. F. S. Stewart and J. B. Eskridge. Mr. Stowe prepared this report.

The Commanders and Directors of the WES during the conduct of the program and the preparation and publication of this report were COL J. L. Cannon, CE, and COL Nelson P. Conover, CE. Technical Director was Mr. F. R. Brown.

CONTENTS

CONVERSION FACTORS, U.S. CUSTOMARY TO METRIC (SI) UNITS OF MEASUREMENT

U.S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
inches	0.0254	metres
feet	0.3048	metres
pounds (mass)	0.4535924	kilograms
pounds (force)	4.448222	newtons
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
pounds (force) per square inch	0.006894757	megapascals
tons (force) per square foot	0.09576052	megapascals
feet per second	0.3048	metres per second
miles	1.609	kilometers

CONCRETE AND ROCK TESTS, REHABILITATION WORK: MARSEILLES DAM, ILLINOIS WATERWAY, CHICAGO DISTRICT

PART I: INTRODUCTION

Location of Study Area

1. The Marseilles Dam is located adjacent to the city of Marseilles on the Illinois River in La Salle County, Illinois, some 70 miles southwest of Chicago. The lock and dam are about 2-1/2 miles* apart with the dam located near mile 247.0 and the lock near 244.6 at the downstream end of the Marseilles Canal.

Background

2. At a meeting held at the offices of the Chicago District Office (CDO) on 15 October 1976, representatives of the Concrete Laboratory (CL) and the Soils and Pavements Laboratory (S&PL), U. S. Army Engineer Waterways Experiment Station (WES), were requested to submit a proposal for work to assist the CDO in the rehabilitation of Marseilles Dam. The following tabulation gives the names and organizations of the attendees at the October meeting:

Fred Paterson	NCD
Terrence Smith	NCD
Terry Goupos	NCD
Carl Pace	WES
Richard Stowe	WES
Mark A. Vispi	WES
I. Juzenas	CDO
George Sanborn	CDO
C. Ruiter	CDO
CPT James R. Van Epps	CDO

NOTE: NCD - North Central Division

* A table of factors for converting U. S. customary units of measurement to metric (SI) units is presented on page 3.

CONVERSION FACTORS, U.S. CUSTOMARY TO METRIC (SI) UNITS OF MEASUREMENT

U.S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
inches	0.0254	metres
feet	0.3048	metres
pounds (mass)	0.4535924	kilograms
pounds (force)	4.448222	newtons
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
pounds (force) per square inch	0.006894757	megapascals
tons (force) per square foot	0.09576052	megapascals
feet per second	0.3048	metres per second
miles	1.609	kilometers

CONCRETE AND ROCK TESTS, REHABILITATION WORK: MARSEILLES DAM, ILLINOIS WATERWAY, CHICAGO DISTRICT

PART I: INTRODUCTION

Location of Study Area

1. The Marseilles Dam is located adjacent to the city of Marseilles on the Illinois River in La Salle County, Illinois, some 70 miles southwest of Chicago. The lock and dam are about 2-1/2 miles* apart with the dam located near mile 247.0 and the lock near 244.6 at the downstream end of the Marseilles Canal.

Background

2. At a meeting held at the offices of the Chicago District Office (CDO) on 15 October 1976, representatives of the Concrete Laboratory (CL) and the Soils and Pavements Laboratory (S&PL), U. S. Army Engineer Waterways Experiment Station (WES), were requested to submit a proposal for work to assist the CDO in the rehabilitation of Marseilles Dam. The following tabulation gives the names and organizations of the attendees at the October meeting:

Fred Paterson	NCD
Terrence Smith	NCD
Terry Soupos	NCD
Carl Pace	WES
Richard Stowe	WES
Mark A. Vispi	WES
I. Juzenas	CDO
George Sanborn	CDO
C. Ruiter	CDO
CPT James R. Van Epps	CDO

NOTE: NCD - North Central Division

* A table of factors for converting U. S. customary units of measurement to metric (SI) units is presented on page 3.

3. As a result of a previous investigation conducted in early 1973,[1] it was concluded that tainter gates sections of the dam failed to meet current overturning criteria requirements. The CDO recommended a plan of proposed rehabilitation of the dam. It was recommended that the tainter gates section be stabilized by installation of grouted, prestressed tendons. The proposed method is similar to the method proposed for stabilizing the tainter gate sections at Brandon Road Dam (see Reference 2, Plate 4), the exception being that at Marseilles the tendons will be put through the piers only. The tendons are designed to meet current overturning requirements under all loading conditions.

4. The WES was asked by CDO staff to review four pertinent reports and then conduct field drilling; to recommend field and laboratory testing of concrete and bedrock; to validate a previous stability analysis; and to analyze, determine, and present a prestressed anchoring system, if necessary. The results of structures work will be presented under separate cover.[3] Copies of four reports, a stability investigation report, a periodic inspection report, a stability analysis report, and a geological foundation report, all pertinent to the lock and dam, were received at the October meeting for review and background information.[1,4,5,6] The reports were reviewed; a proposal was prepared for and approved by CDO.

5. After considering the engineering characteristics of the foundation rock and the material properties as described in the appropriate reports, a recommended in situ and laboratory testing program, as outlined in Table 1, was proposed. Pressure transducer tests were recommended in order to check on the uplift pressures near the boreholes. Televiewer logging was proposed in order to determine the presence and orientations of discontinuities in the foundation so that their effect on the stability and proposed rehabilitation plan could be properly evaluated.

6. A minimum number of characterization properties on representative specimens were recommended to aid in evaluating the consistency of the foundation materials. A minimum number of unconfined compression, triaxial, and tensile tests were recommended on specimens selected to represent each lithologic unit. Strength and stress-strain relations will be obtained from these tests; various moduli and Poisson's ratios could be

calculated for use in structural stability analyses. Direct shear tests were recommended from which peak strength and sliding friction characteristics of portions of the foundation material could be obtained. The types of direct shear tests and the reasons for running them are presented on page 8 in Reference 2. Pullout tests on grouted rebar were recommended from which bond resistance values could be obtained. Further, it was recommended that a detailed petrographic examination be conducted on suspected clay samples.

Objectives

7. The objectives of this study were as follows:

   a. Review available information.

   b. Conduct drilling for field and laboratory testing of concrete and rock.

   c. Make an analysis of tests conducted and a summary of the foundation condition.

   d. Prepare a concrete and rock data appendix for the rehabilitation design memorandum.

Scope

8. The drilling was accomplished using a WES drilling crew, plant, and supplies; floating plant and crane support were furnished by the CDO. A geologist from WES logged the core and preserved it for laboratory testing. Delivery of the core to WES was made in two shipments. The laboratory testing program was initiated after the first shipment of core had been received. Geologic cross sections and sections showing bedrock structural characteristics were developed from available information. These sections were updated as additional information was received. The partial sections containing information from the first shipment and the geological information presented in Reference 6 were used in the initial selection of representative test specimens.

9. The objectives of this study were accomplished by drilling and sampling concrete and rock core; by conducting characterization property

tests consisting of compressive strength, triaxial (including multistage), tensile, direct shear, and rebar pullout tests. The direct shear tests were conducted on concrete-cast-to-rock, intact shale, clay seams in shale, precut, natural joint, open bedding plane, and cross-bedded samples. The shearing load was applied parallel to bedding with the exception of the natural joint and cross-bedded samples. Several suspected clay samples were subjected to a petrographic examination to ascertain mineral content. A borehole pressure transducer device was used in four of the five drilled holes to determine uplift pressures. A borehole televiewer[7] was used in the same holes as was the pressure transducer device in an attempt to obtain information concerning the orientations of discontinuities. The oriented discontinuities in the bedrock were considered in making the foundation appraisal.

PART II: FOUNDATION EXPLORATIONS

Previous Explorations

10. Previous foundation exploration was carried out by the CDO in 1971 for purposes of obtaining a foundation appraisal of the bedrock and backfill of the lock and dam and providing design criteria for use in a structural stability analysis. Three borings were put into the bedrock at the dam, one land boring and two borings just upstream (U/S) of the dam. The borings at the lock are not discussed in this report.

11. The three borings penetrated bedrock. Boring SACM-DAM-2 (U/S of the dam) was drilled to a depth of about 21 ft below the base of the dam. The other U/S boring was taken to 11 ft below the base of the dam, and the land boring was taken to about 34 ft below the dam base. The geologic information obtained from these borings is presented in detail in Reference 6 and was used in obtaining the foundation appraisal reported herein.

Current Drilling

12. Drilling operations began the middle part of December 1976 and were halted in mid-January 1977 at the completion of four of five borings; rising water caused excessive turbulence just downstream (D/S) of the dam, and drilling operations were called off. The fifth boring was drilled in late April and early May 1977. Drilling equipment consisted of an Acker Toredo Mark II skid-mounted rotary drill rig. Six-inch ID diamond core bits and a 4-ft-long double-tube barrel were used to drill the concrete and the bedrock. Access to the drill holes was provided by a marine floating plant with crane. Pertinent information about the borings drilled at Marseilles Dam is tabulated below:

Boring No. MD WES	Section Hole Located	Elevation,* Top of Hole, ft	Depth of Hole, ft	Material Drilled
1-76	Tainter gate pier 1	486.7	89.2	32.9-ft concrete 56.3-ft bedrock
2-77	Tainter gate pier 5	486.7	89.3	34.2-ft concrete 55.1-ft bedrock
3-76	Tainter gate pier 9	486.7	85.9	34.5-ft concrete 51.4-ft bedrock
4-77	U/S Pool	464.1	59.7	59.7-ft bedrock
5-77	D/S Pool	469.0	70.0	1.7-ft concrete 68.4-ft bedrock

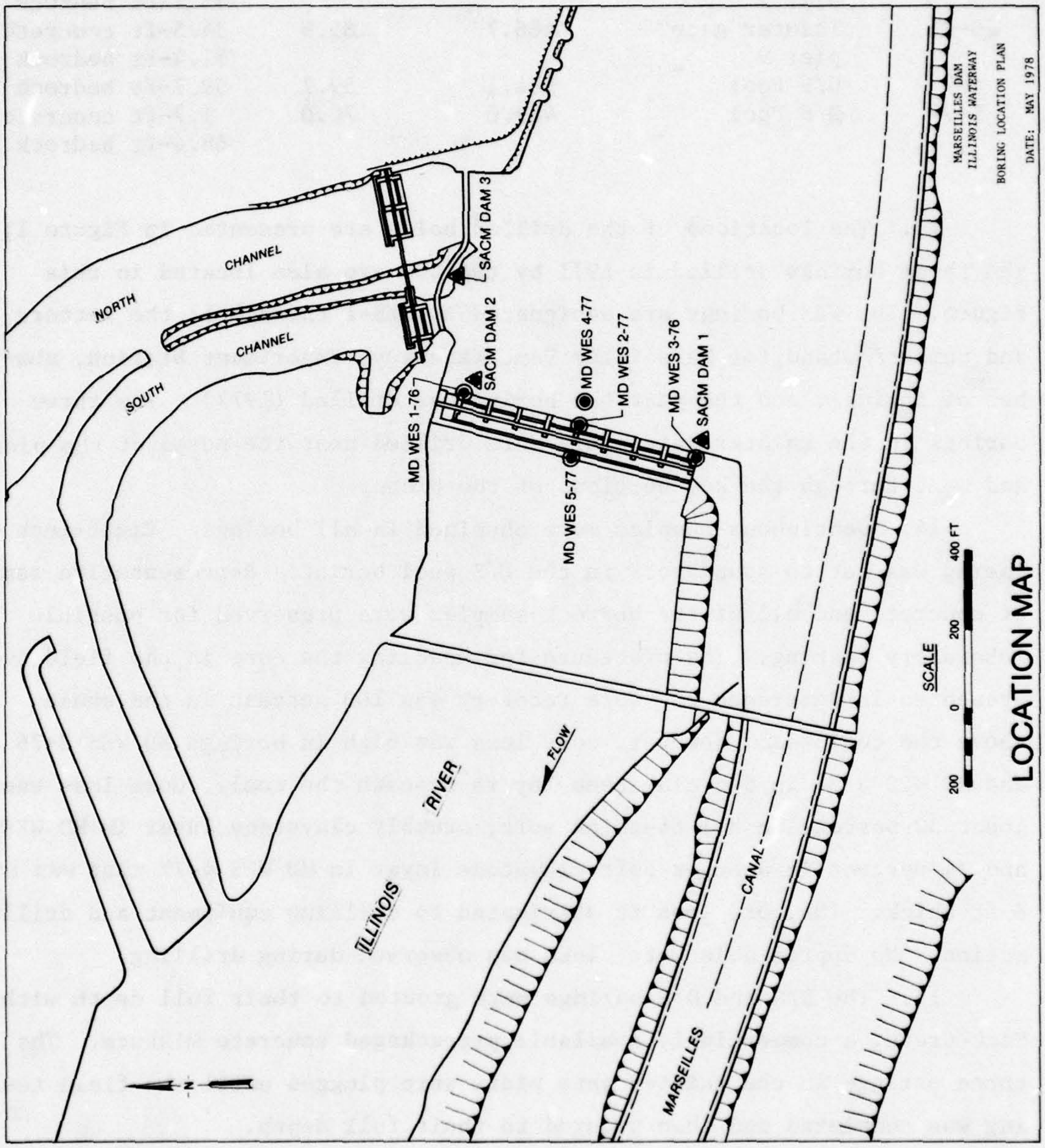
13. The locations of the drilled holes are presented in Figure 1; the three borings drilled in 1971 by the CDO are also located in this figure. The WES borings are designated MD WES-1 through 5; the letters and numbers stand for Marseilles Dam, Waterways Experiment Station, number of borings, and the year the boring was drilled (1977). The three borings in the tainter gate piers were drilled near the noses of the piers and went through the key sections of the piers.

14. Continuous samples were obtained in all borings. Eight-inch casing was set to sound rock in the U/S pool boring. Representative samples of concrete and all of the bedrock samples were preserved for possible laboratory testing. The procedure for handling the core in the field is presented in Reference 2. Core recovery was 100 percent in the shale above the coal seam; however, core loss was high in borings MD WES 3-76 and MD WES 4-77 in two claystone layers beneath the coal. Core loss was about 30 percent in a 1-ft-thick soft, crumbly claystone layer in MD WES 3-76 and 34 percent in another soft claystone layer in MD WES 4-77 that was about 8 ft thick. The core loss is attributed to drilling equipment and drilling action. No appreciable water loss was observed during drilling.

15. The U/S and D/S borings were grouted to their full depth with Sack-Crete, a commercially available prepackaged concrete mixture. The three borings in the tainter gate piers were plugged until the field testing was completed and then grouted to their full depth.

---

* All elevations in this report, including tables, figures, and plates, are in feet referred to mean sea level (msl).


LOCATION MAP
SCALE
200 0 200 400 FT
MARSEILLES DAM
ILLINOIS WATERWAY
BORING LOCATION PLAN
DATE: MAY 1978
FIGURE 1
NORTH CHANNEL
SOUTH CHANNEL
SACM DAM 3
SACM DAM 2
SACM DAM 1
MD WES 1-76
MD WES 4-77
MD WES 2-77
MD WES 3-76
MD WES 5-77
ILLINOIS RIVER
FLOW
MARSEILLES CANAL
N

Pressure Transducer Measurements

16. Measurements of the uplift pressure at the base of the dam were taken in the three holes along the crest of the dam. The drilling rig was used to place a packer at a predetermined elevation. After measurements were taken, the drilling was continued.

17. These measurements were made immediately after the concrete-rock interface had been reached. The technique consisted of mounting a strain-gaged diaphragm pressure transducer in the bottom of a packer and connecting the signal leads to required instrumentation on the top of the dam. With the gage and packer in place, the packer was inflated to seal the hole, and pressure measurements were made continuously for a sufficient period of time to determine the uplift pressure. Approximately one hour was required before a constant pressure was recorded. The uplift pressure can only apply to the area immediately adjacent to the borehole.

Televiewer Logging

18. In the absence of oriented core samples, a borehole televiewer logging tool was used in an attempt to ascertain strikes and dips from the borehole walls. A televiewer was run in the three borings across the crest of the dam and in the D/S hole MD WES 5-77. The full depth of the holes was logged to obtain information on fractures in the concrete and bedrock. The televiewer was run in the holes from a geophysical truck that was placed on a barge. A brief description of the televiewer and how it operates is given in Reference 2. Experience with the device has shown that a smooth, hard surface will reflect better than a rough, soft one. The borewalls in the shale at Marseilles were expected to give less-than-average-quality indications of cracks and voids, especially in the crumbly, soft claystone beneath the coal seam. Less-than-average-quality indications of cracks were obtained from the soft claystone.

Water Analysis

19. Because of the concern that acid water may attack the prestressed tendons, possibly acid water was sampled at elevations just above the coal seam in 4 of the 5 borings. There was speculation that the coal could be generating acid water. A small 12-volt capacity electric pump and flexible hose was used to sample at the prescribed elevations. The volume of water required to prime the pump and fill the length of hose was determined. When sampling for possibly acid water, the calibrated amount of water was allowed to pass through the pump; and the water to be checked was bottled, shipped to WES, and analyzed for chloride, sulfate, sulfide, and pH.

PART III: GEOLOGICAL CHARACTERISTICS

Bedrock Stratigraphy

20. The bedrock beneath and adjacent to the dam consists of sediments of Pennsylvanian age. Nomenclature of these sediments is based upon the average 2.5-ft-thick coal seam encountered during drilling. The seam is correlated with the La Salle No. 2 (referred to as the Colchester No. 2 in Reference 8, p 187). Shale immediately above the coal is assigned to the Francis Creek member of the Carbondale Formation, and the claystone below the coal is assigned to the Spoon Formation; both fall within the Kewanee Group of the Desmoninesian Series. The coal seam and the Francis Creek shale had previously been placed in the McLeansboro Group.[6]

21. The Francis Creek shale overlies the La Salle No. 2 coal, which is a normal, bright-banded coal varying from a fraction of an inch to 2-3.5 ft thick. The La Salle No. 2 overlies the Spoon Formation, which is characterized by underclay, claystone, sandstone, coal, and limestone.

Geologic Cross Sections

22. A log of borings was drawn comprising the five borings drilled at the dam site; see Plate 1. The logs were drawn to show an overview of the bedrock material as well as to assist in the selection of representative test specimens. Three cross sections (see Figure 1 for locations) were prepared using the five borings drilled during the investigation as well as some borings drilled during the 1971 exploration work by the CDO. Two geologic sections numbered A-A and B-B were drawn running about north-south parallel to the dam axis (see Plate 2), and one (see Plate 3) numbered C-C was drawn for the section running about east-west normal to the dam axis. The sections show the continuity of the bedrock beneath and adjacent to the dam and indicate possible weak zones.

23. The concrete-bedrock contact in the three borings through tainter gate piers exhibited no bond; i.e., the contact was loose. Additionally, the first 0.5 ft of bedrock is moderately fractured in two

of these three borings. The bedrock immediately beneath the concrete in the tainter gate piers at Brandon Road Dam was in a similar condition; i.e., a loose concrete-bedrock contact throughout and about 1 ft of highly fractured rock in one boring. Preliminary indications are that the fractured rock could have been caused by blasting during excavation for the dams and/or by rebound after excavation.

24. Most of the foundation rock is shale with a high content of clay minerals. If the rock was fissile, it was called shale and if it lacked fissility, it was called claystone. The shale above the coal is dark gray, medium hard; it contains nodules from 10 to 80 mm consisting of quartz, siderite and pyrite, speckled with small pyrite crystals, and has occasional thin (<2 mm thick) clay seams. The rock is slightly fissile. The shale is from 23 ft in boring 1 to 28 ft thick in boring 3 beneath the base of the tainter gate pier keys; it is 29 to 34 ft thick beneath the base of the pier, assuming a 6-ft-deep key. The lower 12 to 15 ft of the stratum is more clayey than the rock above.

25. The coal was used as the marker bed for correlating the bedrock at the site; it averages 2.5 ft thick. The coal is black and highly fractured, with some fractures filled or partially filled with calcite. The fractures tended to be at 90 degrees to each other and produced blocky particles.

26. Four distinct strata were encountered beneath the coal, and each unit was identified in the five WES borings. Directly under the coal is a dark gray to brownish black claystone with an average thickness of 5.5 ft. The material is clayey, soft, and crumbles easily with moderate hand pressure; it is highly fractured with at least three joint sets with slickensided surfaces. The joints and slickensided features are assumed to be produced by contemporaneous deformation since the joint sets were not present in the rock above this strata. The next strata is a dusky yellow-green claystone with a thickness of 1 to 2 ft. The material is soft, crumbly, and highly fractured and slickensided like the gray claystone above.

27. The sequence of rock below the green claystone and above the last stratum encountered (light gray sandstone) is less continuous between the borings than the sequences of rock above. The rock below the green

claystone is mostly a gray claystone whose color varies from olive gray to dark gray with some organic debris. In boring MD WES 4-77 there is a layer of green claystone and a layer of conglomerate, each about 1 ft thick. Borings MD WES 1-76 and 5-77 contained a limestone layer about 1 ft thick with chert nodules. The physical character of this sequence of rock ranged from very hard, competent limestone to soft, highly fractured claystone.

28. The last rock type encountered is a light gray sandstone, medium-grained, loosely cemented, and very friable. The top one foot normally graded from clayey sandstone to sandstone; maximum thickness drilled was 13 ft in MD WES 1-76. See Appendix A for a more detailed description of the rock types recovered at the drill site.

Bedrock Structural Characteristics

29. The major bedrock structural features relevant to foundations are summarized on the log of borings sheet (see Plate 1); all structural features observed in the core are presented in Appendix A, Plate A1.

30. The bedrock at the dam site is essentially horizontal with a slight dip to the southwest. The dip can be visualized on the log of borings sheet using the coal seam as seen in the three borings in the tainter gate piers. Bedding planes were generally smooth and devoid of irregularities. A number of preexisting bedding plane partings are present in the shale stratum.

31. The shale between the base of the dam and the coal seam (a stratum between 30 and 35 ft thick) contains thin clay seams about 2 mm (1/16 in.) in thickness. The seams are few in number; however, three are considered continuous under the dam (see Plate 1). One clay seam occurred within 3 ft of the base of the tainter gate pier keys of piers 1 and 5.

32. A total of seven fractures that were considered as joints occur in the top shale stratum. The joints are widely spaced and were considered not to be continuous between borings. The inclination of these joints is shown in Appendix A, Plate A1. The 12 to 18 ft of claystone below the coal is highly fractured and contains at least three sets of joints with slickensided surfaces produced by possibly contemporaneous deformation;

the term contemporaneous deformation is used to describe the physical character of the claystone because the inclined jointing did not extend into the coal above nor the sandstone below. The highly fractured and jointed character of the claystone should not effect the stability of the dam.

33. Possible weak zones in the bedrock under the dam are the fractured shale as found in borings MD WES 2-77 and 3-76 and the thin clay seams (<2 mm thick) and preexisting open bedding planes in the shale strata beneath the dam.

PART IV: SELECTION OF TEST SPECIMENS AND TEST PROCEDURES

Cores Received

34. About 108 ft of concrete and 283 ft of rock were received from the 5 borings. Pertinent information concerning the core recovered is presented in Table 2. Two shipments were received during the drilling operation, one in January and one in May of 1977. Selection of test specimens was made shortly after receiving the core, and the specimens were stored in a moist curing room until they were tested.

Selection of Test Specimen

35. The petrographic examination indicated that the deepest concrete deterioration occurred at 1.8 ft. Therefore, concrete test specimens were selected from the first 2 ft of core. For comparison purposes concrete from the mid and bottom portion of the tainter-gate piers was selected for testing. Characterization properties, effect (wet) unit weight (γm), compressional wave velocity (V_p), compressive strength (UC), Young's modulus (E), and Poisson's ratio (ν) were determined or calculated.

36. If an anchorage system was necessary, it was anticipated that the first 25 ft of foundation rock below the tainter-gate pier key would be used for the tendon anchors. Accordingly, only 25 ft of rock beneath the key was extensively tested where feasible test specimens for direct shear tests were obtained in close proximity to the concrete-rock contact. Test assignment locations can be obtained from the tables of test results.

37. There were six types of specimens tested in direct shear: concrete cast on rock, intact (parallel to bedding), precut (rock-on-rock and concrete-on-rock), natural jointed, open bedding plane, and cross-bedded. The natural jointed specimens had jointed surfaces that were quite smooth.

38. A total of 22 cores were tested for compressive strength and were selected throughout the first 25 ft of the borings. The rather large number of strength tests was necessary due to the apparent differences

in soundness of the cores. Specimens for tensile strength (T_s) were selected near the 25-ft depth below the concrete-rock contact of the key where the greatest tensile stress would be imparted to the foundation by the anchors of the prestressed tendons, should prestressing by tendons be necessary.

Test Procedures

39. The test procedures used to conduct laboratory tests during this investigation were the same ones used for conducting the laboratory tests for the Brandon Road Dam stabilization program; see Reference 2. One additional test, rebar pullout resistance, was conducted during this study that was not conducted for the Brandon Road Dam stabilization project.

40. The rebar pullout resistant test is sketched in Figure 2. A 6-in.-diameter by 12-in.-long shale core was placed upright in a 12-in.-high by 30-in.-diameter mold. A concrete mixture having the average compressive strength of the rock (the 2 zones of rock within the first 25 ft of shale had strengths of 430 and 2160 psi) was placed into the mold embedding the core to its full length. The concrete portion served two purposes. First, it acted as a resistance block allowing the rebar to be pulled, and second, it served as a host material in case the core instead of the rebar was pulled out.

41. After the concrete had cured and attained the desired strength, a 1-in.-diameter hole was drilled in the center of the shale core. A diamond thin-wall bit was used resulting in a smooth-walled borehole. A No. 4 rebar was grouted the full depth of the core using a commercially available premix grout for anchoring bolts and dowels. Grout strength at test time was about 4000 psi. After the grout attained desired strength, the bars were pulled using the setup depicted in Figure 2. Total weight of the suspended specimen was taken into account in calculating the bond stress.

Petrographic Examination

42. The various specimens selected for examination and the test procedures used to conduct the petrographic work are presented in

Appendix A of this report. Photographs of the core that were taken in the field have been assembled in a loose-leaf notebook. One set is on file at the CDO and one set at WES. The photo notebook is Exhibit 1 to this report.

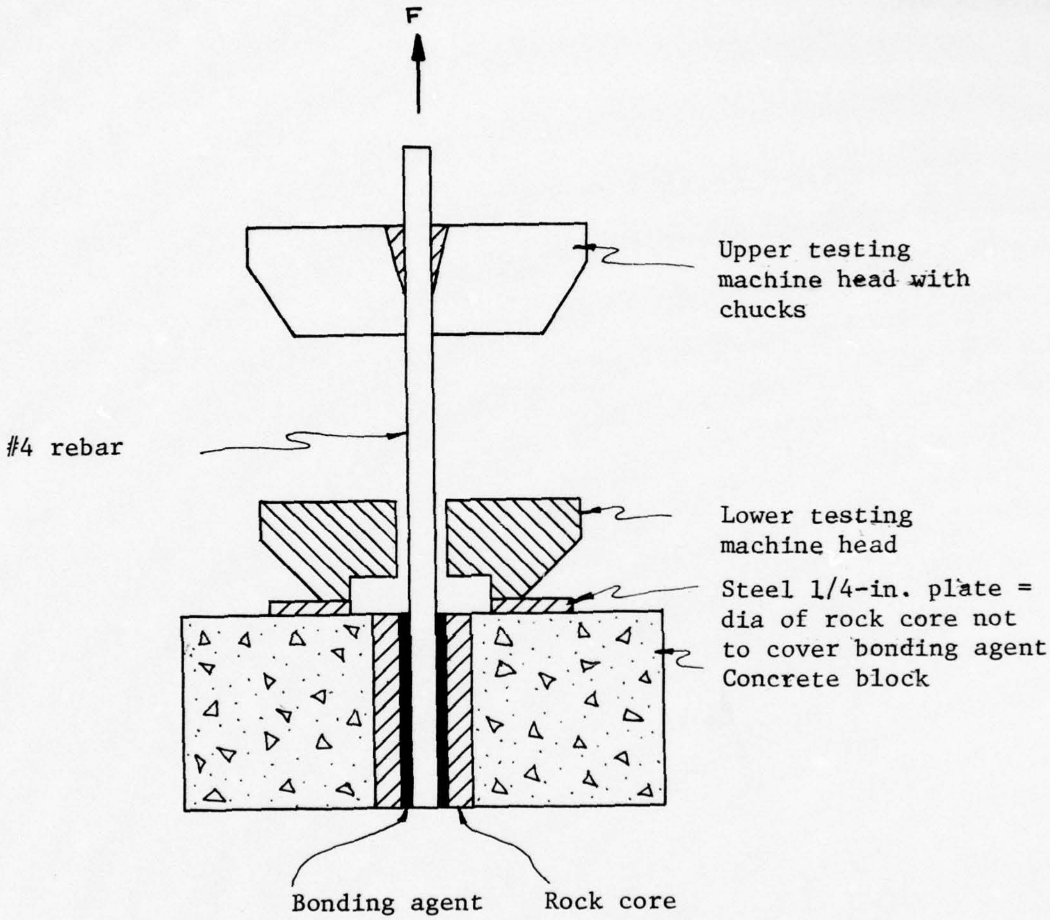


Figure 2 . Section showing test configuration for the reinforcing bar pullout tests.

PART V: TEST RESULTS AND ANALYSIS

Pressure Transducer Measurements

43. The results of the pressure transducer tests are tabulated below. Refer to Figure 1 for boring locations.

Drill Hole No.	Tailwater H_1, ft	Headwater H_2, ft	Pressure Measured in Boring psi	Head in Boring ft	Uplift at Foundation Contact Plane, ft
MD WES-1-76	17.5	29.8	9.64	22.27	29.50
MD WES-2-77	17.8	30.5	10.95	25.28	30.20
MD WES-3-76	18.4	31.3	13.43	31.00	31.00

"The uplift pressure at any point under the structure will be tailwater pressure plus the pressure measured as an ordinate from tailwater to the hydraulic gradient between upper and lower pool."[9] The uplift at the foundation contact plane was calculated assuming that the hydraulic gradient varies, as a straight line, from headwater to tailwater. The following equation was used to calculate the uplift at the foundation contact plane:

$$\text{Uplift} = H_2 + \frac{X}{L}(H_2 - H_1) \quad \ldots \quad 1$$

where H_1 is tailwater, H_2 is headwater, X is distance from toe and L is length of base; see Reference 9.

EM 1110-2-2200 states that where no provisions for uplift reduction have been made, the hydraulic gradient will be assumed to vary, as a straight line, from headwater to tailwater. Although drains are shown on the contact drawings, the drainage system under the dam is assumed not to be 100 percent effective; i.e., the system is partially clogged. The indication that the system is less than 100 percent effective is the close

correlation between the calculated uplift and the measured head in the three borings. The borehole head was 75, 84, and 100 percent of the calculated hydraulic gradient for borings 1, 2, and 3, respectively.

44. As mentioned earlier, the uplift pressure measurements were conducted as a matter of interest; however, they do indicate improper drainage under the dam.

Discontinuities from Core Logs and Televiewer

45. Borehole televiewer surveys were run in borings 1, 2, 3, and 5; i.e., the three borings along the axis of the dam and the one boring behind the dam. The televiewer logs provide records of the borings surveyed and are presented in Appendix B along with an interpretation of the logs. The logs were made by a commercial company and are considered fair for the foundation material at Marseilles Dam.

46. Prior to analysis of the televiewer logs, core logs and core photos were consulted with the result that little geologic structure was indicated in the subsurface to the depth of the coal seam. The geologic structure considered important for purposes of stability analysis includes joints or bedding planes dipping greater than 10 degrees. Only nine fractures in the first 25 ft of bedrock below the pier key, most of which were short discontinuous fractures, were described in the core logs of the five borings. Seven of these fractures were considered joints, and only three of the features were correlated with similar features on the televiewer logs. The bedrock below the coal contained 68 fractures, most of which were considered as joints. Three joint sets were distinguished having ranges of 10 to 15 degrees, 20 to 30 degrees, and 45 to 53 degrees. The majority of the 68 fracture surfaces contained slickensides and were probably produced by contemporaneous deformation (folding and faulting that takes place while rocks are being deposited). Three of the 68 fractures were distinguishable on the televiewer logs. The six fractures discernible on the televiewer logs are tabulated below.

Drill Hole No.	Elevation ft	Strike and Dip
MD WES-1-76	438.7	N70°W, 13°NE
MD WES-1-76	422.7	N80°E, 20°SSE
MD WES-1-76	411.2	N48°E, 34°SE
MD WES-1-76	408.7	N48°E, 34°SE
MD WES-2-77	435.8	N90°D, 39°S
MD WES-2-77	428.5	N1°E, 31°E

47. The majority of the other features visible on the televiewer logs are probably horizontal bedding-related structures such as vugs, primarily solution cavities, zones of soft material such as clay and shale that washed out during drilling, and light and dark banding by clay beds. The fractures occurring in the boreholes were too scattered to establish the existence of joint sets or set orientations. The joint planes detected adjacent to the base of the dam, if they participated in a failure, could act as an inclined plane on which a horizontal failure could daylight. Because the joints are few in number and were not detected as being continuous between borings, they would participate in local failures. The determination of the orientation of the few joints close to the base were not obtained with the televiewer. However, dip angle and shear strength along the joints should be considered when making the stability analysis.

48. In all three of the borings across the dam, the concrete-bedrock contact was loose. This fact should be seriously considered when making the stability analysis.

Petrographic Examination

49. Most of the concrete examined was in good condition. The concrete cores remained intact and only occasional breaks were observed. However, none of the breaks except those near the top of the structure were considered to be old breaks. Deterioration of concrete was observed in cores from borings 1 and 2, limited to the near-surface concrete to

depths of 1.5 ft and 1.8 ft, respectively. Concrete from boring 5 is underwater during subfreezing conditions and therefore has limited exposure to freezing and thawing. Boring 3 is located close to an abutment and contained no detectable deteriorated concrete. The deterioration was caused by cycles of freezing and thawing.

50. The concrete consisted of 1-1/2-in. maximum size natural gravel composed of carbonate and igneous coarse aggregate particles. The fine aggregate was a natural siliceous sand. No evidence of alkali-aggregate reactions was observed.

51. The concrete was well consolidated and did not show signs of segregation. This structure was built before the use of air entrainment in concrete. There were some small entrapped air voids found throughout the concrete; however, these void areas will have no effect on the structure as is, or if it is prestressed with steel tendons.

52. Most of the foundation rock is shale with a high content of clay minerals. The rock above the Colchester No. 2 coal was called shale because it was fissile and the rock, except the sandstone, below the coal was called claystone because it lacked fissility.

53. A detailed description of each rock stratum encountered during drilling is presented in Appendix A. Therefore, just a brief description of the rock sequence is presented here. Below the concrete is a medium dark gray shale. All of the rock examined from this interval was composed of quartz, plagioclase feldspar, and siderite as the nonclay constituents. The clay minerals present in this rock were clay-mica, kaolinite, and chlorite. Thin clay seams are present. Coal is below this unit and in large part highly fractured. The fractures tended to be at 90 degrees to one another which produced blocky pieces. Three distinct claystone strata are present below the coal. The sequence is a dark gray to brownish black claystone, a dusky yellow-green claystone, and a mostly gray claystone varying from olive gray to dark gray. These three strata are highly fractured and contain at least three sets of joints with slickensided surfaces produced by possible contemporaneous deformation. The last rock drilled is a friable, medium-grained, light gray sandstone.

Water Analysis

54. Water samples were recovered from four of the five borings. The results of the tests run on the samples are tabulated below.

Drill Hole No.	Elevation ft	Chloride ppm	Sulfate ppm	Sulfide ppm	pH ppm
MD WES-2-77	426.0	57.8	50.0	0.1	7.8
MD WES-3-76	424.0	50.6	44.2	<0.1	7.7
MD WES-4-77	425.0	62.1	61.6	<0.1	7.7
MD WES-5-77	427.0	76.7	96.0	<0.1	7.9

55. The water was not found to contain any large amounts of the above anions, and the pH indicates that the water is slightly alkaline. The water from these elevations should not be corrosive to steel nor to a cementitious grout used to anchor steel tendons in the shale or coal.

Characterization Properties

Concrete

56. The results of the characterization property tests are presented in Table 3. The average value, the standard deviation, the range and the number of tests for the concrete are tabulated below. The

	Effective Unit Wt γ_m, lb/ft^3	Dry Unit Wt γ_d, lb/ft^3	Water Content w, %	Compressional Wave Velocity V_p, fps	Compressive Strength UC, psi	Tensile Splitting Strength T_s, psi
Average	155.2	147.0	5.6	17,210	9260	660
Standard Deviation	0.9	1.6	0.6	399	803	25
Range	3.1	4.9	1.9	1,305	2190	50
No. of test	9	9	12	9	9	3

properties of the concrete core containing portions of deteriorated concrete were similar to core without deteriorated concrete. Little damaged concrete was included in two of the test specimens. No distinction in physical properties between frost damaged and sound concrete is made.

57. The average unit weights, wave velocity, and compressive strength are indicative of sound concrete. These properties are quite consistent throughout the full length of recovered core indicating good consolidation and quality control at the time of placement. The concrete is considered structurally adequate for its intended purpose.

58. Selected physical properties are located by elevation on cross-section sheets having the same format as the geologic cross section described earlier (Plate 4). The plot gives a visual comparison of property data for the length of concrete and bedrock core drilled and tested. In general, the γ_m, V_p, and UC of the concrete increases with depth. The greatest concrete strength and velocity were 10,280 psi and 17,800 fps, respectively, for concrete near the bottom of the tainter-gate piers. The average strength and velocity of the concrete in the Marseilles Dam is higher than similar values of concrete in the Brandon Road Dam; i.e., the strength and velocity was 38 and 10 percent higher, respectively.

Bedrock

59. The results of the characterization property tests of the bedrock are presented in Table 4. The average value, the standard deviation, the range, and the number of tests are tabulated below. The 23- to 28-ft thick shale stratum just beneath the pier key contains two distinct zones based upon the characterization tests. The shale is tabulated in two groups to emphasize this fact. The more competent upper zone (Zone 1) is about 12 ft thick, and the less competent lower zone (Zone 2) is about 14 ft thick; see Plate 4 for exact thickness of zones.

	Effective Unit Wt γ_m, lb/ft^3	Dry Unit Wt γ_d, lb/ft^3	Water Content w, %	Compressional Wave Velocity V_p, fps	Compressive Strength UC, psi	Tensile Splitting Strength T_s, psi
Zone 1 Shale 12 ft thick below pier key						
Average	158.9	150.5	6.4	8385	2160	120
Standard Deviation	1.2	1.4	0.9	685	270	26
Range	2.0	2.8	2.6	1215	780	55
No. of tests	3	3	10	3	7	4
Zone 2 Shale 14 ft thick just below Zone 1						
Average	148.9	138.7	7.4	6820	430	--
Standard Deviation	4.4	6.2	2.2	255	214	--
Range	10.5	14.5	5.4	470	550	--
No. of tests	5	5	5	3	6	--
Claystone						
Average	140.8	121.9	10.1	--	60	--
Standard Deviation	7.4	--	2.7	--	32	--
Range	13.0		4.7	--	70	--
No. of tests	3	1	3	--	4	--
Coal						
Average	94.3	84.0	12.3	--	2570	--
No. of tests	1	1	1		1	

60. Although a small sampling was considered, it is believed that the above values are representative because the relatively small standard deviations and ranges for most of the tests indicate consistency. An analysis of the characterization properties presented is by rock type.

a. Shale, Zone 1. The unit weights are consistent and reasonable for a well compacted shale. The γ_m and γ_d (158.9 and 150.5 lb/ft^3) correlate quite well with similar values, γ_s = 155 lb/ft^3 and γ_d = 150.0 lb/ft^3, reported in Reference 6. The Zone 2 shale and the claystone have lower unit weights. The V_p ranged from a low 7960 to 9175 fps and in general, are not in good agreement with V_p data reported earlier.[6] The previously reported V_p data ranged between 5795 and 6320 fps. The V_p data might be useful in correlations with in situ seismic velocities if such were available. The compressive strength for Zone 1 shale is reasonable; these test results indicate that the shale is uniform in consistency. The average strength is 2160 psi, with a relatively low range of 780 psi. Our strength values in no way compare to similar values reported earlier.[6] The report presents an average value of 4600 psi for the lock and dam and one value for the dam of 5795 psi. A close review of Reference 6 reveals that water contents were not obtained; had they been, some correlation in strength might have been made. Another factor that makes it difficult to correlate the strengths is that the previous specimens were 1 in. in diameter, while ours were 6 in. in diameter. Specimen size effects strength; for example, large specimen contains more natural discontinuities. Reference 6 points this fact out on p 27.

> "this size sample is not considered ideal and could have unduly influenced the results."

The γ_m = 158.9 lb/ft^3 and the UC = 2160 psi are recommended as design parameters for the Zone 1 shale.

b. Shale, Zone 2. The average unit weights are about 93 percent of the Zone 1 shale and have a larger standard deviation and range. The average V_p is about 81 percent of the Zone 1 shale, with a smaller standard deviation and range than the shale above. The unit weight and V_p indicate a consistent material. The compressive strength is considerably lower than the Zone 1 shale; it is only 20 percent of the shale in Zone 1. Reference 6 points out that, due to heavy slacking caused by the samples drying out, possible weak materials were not tested. This shale and the claystone below the coal are weak materials that were not sampled and tested previously.[6] The design of any anchorage system should take into account the large difference between the Zone 1 and Zone 2 shales. The bearing capacity and deflection of these two units should be seriously considered. The γ_m = 148.9 lb/ft^3 and UC = 430 psi are recommended as design parameters for the Zone 2 shale.

c. Coal. Only one sample of coal was obtained for testing. The characterization properties are presented here for

comparison purposes. The strength of the one coal specimen is similar to the Zone 1 shale.

If an anchorage system were designed for the dam and the anchorage zone were near or in the coal, then additional characterization properties, namely compressive strength and tensile strength, should be obtained. The in situ rock at and near the end of the anchorage zone would be in tension, and this value would be needed in a stress analysis of the anchorage zone.

Engineering Design Properties

Modulus of elasticity and Poisson's ratio

61. Results of the modulus of elasticity and Poisson's ratio tests are presented in Table 3 for the concrete and in Table 5 for the bedrock. Selected values are also presented in Plate 4 at the emblematic location from which the test specimens were taken. The stress-strain relation recorded for a typical concrete core and for rock cores is presented in Plate 5 and Plates 6 through 9, respectively. Composite plots for the Zones 1 and 2 shale and claystone are present in Plates 6, 7, and 9, respectively, and a single curve for coal is plotted on Plate 8. The E for the rock was calculated as a tangent value at 50 percent of ultimate load; the E for the concrete was calculated in accordance with CRD-C 19-75.[10] Most of all the rock stress-axial strain curves were concave upwards initially and then became linear. Poisson's ratio was calculated at the same stress levels as were the E's.

62. The average E and ν for the concrete is 5.18×10^6 psi and 0.20. In general the modulus increased with depth in the tainter-gate piers. The E and ν values are indicative of sound concrete.

63. The modulus of the bedrock changes considerably with depth. The Zone 1 shale has an average E and ν of 0.14×10^6 psi and 0.46, respectively. The Zone 2 shale has an average modulus of only 21 percent of the Zone 1 shale, i.e., $E = 0.03 \times 10^6$ psi. This large difference in deformation of the two shales should be considered in any anchorage design system. The average ν of the Zone 2 shale is 0.35. The modulus of the

coal (only one specimen could be obtained for testing) iw quite high compared to the shales; the E = 0.4×10^6 psi. The claystone beneath the coal seam has a relatively low modulus (0.01×10^6 psi) compared to the rocks above. The average moduli for the four strata are recommended as design parameters for any anchorage system used at the dam.

Triaxial

64. The stress values obtained during the standard triaxial testing are presented in Table 5 and plotted on Plate 10 using the p-q diagram. See Reference 11 for an explanation of p-q diagram and how to relate the parameters α and a to ϕ and c that are readily obtained from a Mohr diagram. Only one series of tests (σ_3 = 27, 55, and 111 psi) was conducted. The series was run for purposes of comparing the shear strength results from the triaxial tests with similar results from the direct shear tests.

65. The strength results from these tests do not appear reasonable; the c = 15 psi (1.1 tsf) and ϕ is 71 degrees. The ϕ is much higher than expected for a medium hard shale (compressive strength about 2100 psi). A plausible explanation for the high phi value is that the specimen tested at σ_3 = 111 psi was composed of a much harder shale than were the other two specimens (tested at σ_3 = 27 and 55 psi). Upon reexamination of all the core, the harder shale was found to occur only locally. Another test on a representative specimen was not run because the triaxial series was conducted for comparison purposes. The comparison, therefore, between the triaxial and the direct shear tests was not made.

66. A multistage triaxial test was conducted using a piece of concrete and shale from hole MD WES-1-76. The results are given in Plate 11. The test method used is quite similar to a standard triaxial test method.

67. The results of the multistage test were used to calculate the shearing stress (τ) across an established surface for various values of the normal stress (σ_n). The preselected surface was 45 degrees from the horizontal. With this information, the coefficient of friction (ϕ_j) on the surface was determined. When the principal stresses are known and the τ and the σ_n on a surface at an angle θ with respect to the principal plane are required, the following equations can be used to calculate these stresses:

$$\sigma_n = \frac{\sigma_1 + \sigma_3}{2} + \frac{\sigma_1 - \sigma_3}{2} \cos 2\theta \qquad (1)$$

$$\tau = \frac{\sigma_1 - \sigma_3}{2} \sin 2\theta \qquad (2)$$

The values of σ_n and τ and then plotted, and values of ϕ_j and c determined.

68. The stress circles for the six loading stages were plotted. The τ and σ_n were plotted on the stress circles and connected to form the strength envelope for the sawed surface. The equation of the stength envelope is as follows:

$$\tau = 0.0 + 0.5774\ \sigma_n$$

The equation yields an angle of shearing resistance of 30.0 degrees and a zero cohesion. Due to the loose contact between the concrete and rock described earlier, the *sliding friction* value from the multistage triaxial test is recommended for computing structural stability for that portion directly under the dam.

Peak shear strength

69. Two types of direct-shear tests were conducted to ascertain peak strength of intact specimens and sliding friction characteristics of discontinuous specimens. Peak strengths were measured for the shale containing a concrete-rock interface, intact shales, and cross-bedded intact shales. Sliding friction properties were measured for specimens of shale along either precut surfaces, naturally occurring joints, or preexisting open bedding planes. Residual shear strength parameters were not obtained for any of the direct shear tests. The shear strength values from the precut samples should closely approach a residual value for the intact shale. The stress values for the direct-shear tests are presented in Table 6 and all but one of the failure envelopes are plotted on Plates 12 and 13.

70. Three direct shear tests were conducted along the interface of concrete and rock. The three specimens were prepared by casting concrete to the natural bedding planes. A failure envelope is not presented for these test specimens because a negative cohesion was obtained when the stress results were analyzed. A rerun of one or more specimens was deemed not warranted because the concrete bonded to bedrock would not control stability. Evaluation of bedrock structural characteristics indicates that other conditions would control stability of the dam; namely, the thin clay seams within the Zone 1 shale.

71. Tests of intact shales (Zone 1 shale) were performed on two types of specimens, representative medium strength shale and the same strength shale containing a thin clay seam. The test results showed relatively little scatter and produced smooth envelopes, indicating reliability in the testing technique employed. From Plate 12, it will be noted that there is a large difference in the angles of friction between the specimens containing the clay seam and the specimens without the seam; ϕ's of 17 and 19 degrees were obtained for specimens with clay seams and ϕ's of 45 and 47 degrees were obtained for specimens without the seam. All specimens were tested parallel to bedding. A previous study[6] recommends shear strength parameters for intact shale of c = 2.9 tsf and ϕ = 47 degrees; these data correlate well with the data obtained during this study for intact shale without clay seams. It is recommended that the shear strength parameters for the shale with clay seams (c = 2.7 tsf and ϕ = 17 degrees) be used as design values for intact shale. The clay seams are continuous over the foundation and occur close to the concrete-bedrock contact in the three borings through the dam.

72. The cross-bedded specimens were tested at an angle of 45 degrees to bedding. The shear strength values were c = 2.63 tsf and ϕ = 42 degrees; these values are very close to similar values obtained from the intact shale without the clay seams.

Sliding friction

73. The rock-on-rock friction tests were of three kinds: shear of precut rock surfaces, natural joints, and preexisting open bedding planes. All tests were conducted on shale cores. The respective test results are labeled and presented in Plate 13.

74. The precut specimens normally produced lower bound shear strength parameters. The c and ϕ for the precut specimens are 3.33 tsf and 27 degrees, respectively, which is slightly higher than similar values for the specimens containing an open bedding plane. A c = 0.68 tsf and a ϕ = 25 degrees were obtained for the specimens containing open bedding planes. The precut surfaces can be considered slightly rougher than the open bedding plane surfaces because of the saw marks contained on the surfaces of the precut specimens. The surfaces of the specimens with open bedding planes did appear relatively smooth prior to testing. The open bedding planes occurred at random locations in the foundation, however, they are not considered the structural feature likely to control stability within the foundation. The clay seams in the Zone 1 shale are considered the controlling feature.

75. The c and ϕ for the natural jointed features are 0.64 tsf and 38 degrees. The jointed surfaces were not smooth like the open bedding plane surfaces, but slightly irregular in that they were gently undulating. As mentioned in paragraph 45, the joints observed in the shale strata beneath the dam could participate in a failure by acting as an inclined plane, therefore, it is recommended that the shear strength parameters on natural joints be considered when making a stability analysis.

Rebar pullout resistance

76. The bond strength calculated from the rebar pullout tests is presented in Table 5. The average bond strength for the Zone 1 and Zone 2 shale were 216 and 31 psi. These strengths represent the bond developed between the cementitious grout used to embed the rebar in the shale and the shale itself. Both values are recommended to be used in the design of any anchorage system at the dam.

PART VI: SUMMARY OF CONCRETE AND FOUNDATION CONDITION AND RECOMMENDED STABILITY AND DESIGN VALUES

Concrete Condition

77. Only three vertical borings were put down in the noses of three separate piers of the dam; therefore, the general condition of the concrete over the dam could not be ascertained. The concrete test results are indicative of the condition of the concrete in the nose area of the piers.

78. Maximum frost damage reaches to a depth of 1.8 ft in pier 2. The concrete within the first 1.8 ft of the nose sections of the taintergate piers, which has deteriorated due to frost action, should be removed before any anchorage system is started. Continual exposure to frost action will cause additional concrete damage at an increasing rate if the concrete remains in place.

79. The concrete below 1.8 ft is considered to be adequate for its intended purpose; it is sound, having a compressive strength in excess of 8000 psi and an elastic modulus greater than 4.2×10^6 psi. The concrete was well consolidated and did not show signs of segregation. This structure was built before the use of air entrainment in concrete. Some local areas of entrapped air voids were present throughout the core; however, these void areas will have no effect on the structure as is, or if it is prestressed with steel tendons.

Foundation Condition

Bedrock stratigraphy

80. The foundation rock beneath the dam consists of sediments of Pennsylvanian age. The La Salle No. 2 coal was used as a marker bed and used to correlate the rock. Shale assigned to the Francis Creek member of the Carbondale Formation is present above the coal. Claystone with limestone and underclay assigned to the Spoon Formation is present below the coal. Both the Francis Creek and Carbondale Formation are within the Kewanee Group of the Desmoninesian Series.

Geologic cross sections

81. The cross sections indicate the extent of deteriorated concrete, the extent of the strata, the continuity of the bedrock beneath and adjacent to the dam, and indicate possible weak zones.

82. The column of rock below the dam consists of shale, coal, and claystone, with underclay, limestone, and sandstone. The shale is dark gray; medium hard; contains nodules of quartz, siderite, and pyrite; and contains occasional continuous thin (<2 mm thick) clay seam. The shale is about 32 ft thick beneath the dam, with the bottom 12 to 15 ft being more clayey. The shale is divided into two zones based mainly upon large differences in physical properties, the lower portion (12 to 15 ft) being the weaker material. The coal is typical blocky fracture material having a thickness of about 2.5 ft. Four distinct strata were encountered below the coal. Directly under the coal is a dark gray to brownish black claystone about 5.5 ft thick. The next stratum is a dusky yellow-green claystone 1 to 2 ft thick. The next stratum is mostly a gray claystone varying in color from olive gray to dark gray; a few discontinuous limestone layers were observed. The last stratum encountered is a light gray sandstone, medium grained, loosely cemented, and quite friable. All strata can be traced across the dam site.

83. The rock between the coal and the sandstone is highly fractured and slickensided; considerable inclined fracturing is present. Because the inclined jointing does not extend into the coal nor the sandstone, this evidence of small scale deformation, jointing, and faulting is considered to be contemporaneous deformation. The water analysis shows that water from the coal seam is not corrosive to steel or concrete.

Bedrock structural characteristics

84. The concrete-bedrock contact in the three borings through tainter-gate piers exhibited no bond; i.e., the contact was loose. The loose contact is considered a probable potential sliding plane at the base of the dam. In two of the borings, the first 0.5 ft of rock is moderately fractured. It is assumed that the fractured rock could have been caused by blasting during excavation, rebound due to excavation, or a combination of both. Similar fractured rock was observed at Brandon

Road Dam. The prestressing of tendons should not affect the interface of the concrete bedrock and fractured rock other than by tightening it.

85. The bedrock at the dam site is essentially horizontal, with a slight dip to the southwest. Bedding planes are generally smooth. Randomly located, short duration, open bedding planes are present in the shale strata. The shale contains thin clay seams about 2 mm thick; although few in number, three are continuous under the dam. The clay seams are the major structural feature in the foundation. If a shear failure occurs within the foundation, it is likely that it will be along a clay seam or seams.

86. A minor amount of jointing is present just beneath the dam and is not considered continuous between borings. Locally the joints could participate in a failure by acting as an inclined plane on which a horizontal failure could daylight. The fractures and slickensided zone below the coal should not affect the stability of the dam.

Recommended Stability and Design Values

87. Concrete, rock type, and the various structural features described herein should be considered when formulating the stability analysis and design parameters of the anchorage system. Guidance is presented in the following tabulation for proper choice of design parameters.

	Concrete	Zone 1 Shale	Zone 2 Shale	Coal	Claystone
Characterization Properties					
Effective (wet) unit weight, lb/ft^3	155.2	158.9	148.9	94.3	140.8
Dry unit weight, lb/ft^3	147.0	150.5	138.7	84.0	121.9
Compressive strength, psi	9260	2160	430	2570	60
Tensile strength, psi	660	120	--	--	--
Shear strength					
Concrete-on-rock	--	c=1.06 tsf $\phi=67^o$	--	--	--

(Continued)

	Concrete	Zone 1 Shale	Zone 2 Shale	Coal	Claystone
Intact*	--	c=1.03 tsf $\phi=45^o$	--	--	--
Intact,* thin clay seam	--	c=2.69 tsf $\phi=17^o$	--	--	--
Precut, rock-on-rock	--	c=3.55 tsf $\phi=24^o$	--	--	--
Precut, concrete-on-rock	--	c=0.0 $\phi=30^o$	--	--	--
Natural joint	--	c=0.64 tsf $\phi=38^o$	--	--	--
Open bedding plane	--	c=0.65 tsf $\phi=25^o$	--	--	--
Cross bed	--	c=2.63 tsf $\phi=42^o$	--	--	--
Bond strength, psi		216	31		
Modulus of elasticity psi x 10^6	5.18	0.14	0.03	0.4	0.01
Poisson's ratio	0.20	0.46	0.35	--	--
Shear modulus psi x 10^6	2.16	0.05	0.01	--	--

* Parallel to bedding

REFERENCES

1. U. S. Army Engineer District, Chicago, "Stability Investigation, Existing Marseilles Lock and Dam, Illinois Waterway," Jul 1973.
2. Stowe, R. L., "Concrete and Rock Test, Rehabilitation Work, Brandon Road Dam, Illinois Waterway, Chicago District," Miscellaneous Paper C-78-4, May 1978, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.
3. Pace, Carl E., "Stability Analysis, Marseilles Dam, Illinois Waterway, Chicago District" (in preparation), U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.
4. U. S. Army Engineer District, Chicago, "Periodic Inspection; Report No. 2, Marseilles Lock and Dam, Illinois Waterway," Oct 1973.
5. __________, "Appendix E, Stability Analysis of Dam Masonry, Marseilles Lock and Dam," Jul 1973.
6. __________, "Appendix A, Soils and Geology for Structural Stability Analysis, Marseilles Lock and Dam, Illinois Waterway," Sep 1972, Revised Jul 1973.
7. Zemanek, J., et al., "The Borehole Televiewer - A New Logging Concept for Fracture Location and Other Types of Borehole Inspection," Journal of Petroleum Technology, Jun 1969 (Reprinted).
8. William, H., et al., "Handbook of Illinois Stratigraphy," Bulletin 95, 1975, Illinois State Geological Survey.
9. "Gravity Dam Design," EM 1110-2-2200, Change 2, Nov 1960.
10. U. S. Army Engineer Waterways Experiment Station, CE, "Handbook for Concrete and Cement," Aug 1949 (with quarterly supplements), Vicksburg, Miss.
11. Lambe, T. W., and Whitman, R. V., Soil Mechanics, Wiley & Sons, New York, 1969, pp 137-143.

Table 1

Recommended In-Situ and Laboratory Testing Program

Rehabilitation Work, Marseilles Dam

Illinois Waterway, Chicago District

Field

Test	Remarks
Downhole Pressure Transducer	Ascertain uplift pressure adjacent to borehole
Downhole Televiewer	Determine orientation of discontinuities

Laboratory

Test	Remarks
Index Properties	Wet and dry unit weights; moisture content; compressional wave velocity
Unconfined Compression	Stress-strain diagram, Young's modulus of elasticity, and Poisson's ratio
Standard & Multistage Triaxial	Undrained tests of 100, 500, and 1000-psi confining pressure, shear modulus and stress-strain diagrams, Young's modulus, and Poisson's ratio
Tensile Splitting	Strength only
Direct Shear:	
Intact Rock	Peak strength
Intact Shales, Friable Rock or Filled-Partings	Peak strength
Precut and Jointed Surfaces	Sliding friction (three-stage multi-loading test)
Concrete-Rock Interface	Peak strength
Detailed Petrographic Examination	Suspected clay specimens
Pullout	Bond resistance

Table 2

WES Cores from Marseilles Dam, Illinois Waterway, Chicago District

WES Reference	Drill Hole No.	Date Rec'd	Core Diam, in.	Box No.	Depth, ft	Elevation, ft msl: Depth Intervals	Elevation, ft msl: Top of Hole	Remarks
CH1-11 CON-1(A)	MD WES-1	1-11-77	6	1 of 18	0.0 - 4.8	486.7 - 481.9	486.7	
(B)				2 of 18	11.2 - 16.0	475.5 - 470.7		
(C)				3 of 18	20.7 - 25.0	466.0 - 461.7		
CH1-11 CON-1(D)				4 of 18	32.0 - 35.8	454.7 450.9		
CH1-11 DC-1(A)				5 of 18	35.8 - 39.5	450.9 - 447.2		
(B)				6 of 18	39.5 - 43.7	447.2 - 443.0		
(C)				7 of 18	43.7 - 47.7	443.0 - 439.0		
(D)				8 of 18	47.7 - 51.7	439.0 - 435.0		
(E)				9 of 18	51.7 - 55.7	435.0 - 431.1		
(F)				10 of 18	55.6 - 57.2	431.1 - 429.5		
(G)				11 of 18	57.2 - 61.2	429.5 - 425.5		
(H)				12 of 18	61.2 - 65.8	425.5 - 420.9		
(I)				13 of 18	65.8 - 69.5	420.9 - 417.2		
(J)				14 of 18	69.5 - 73.5	417.2 - 413.2		
(K)				15 of 18	73.5 - 77.9	413.2 - 408.8		
(L)				16 of 18	77.9 - 80.7	408.8 - 406.0		
(M)				17 of 18	80.7 - 84.0	406.0 - 402.7		
CH1-11 DC-1(N)	MD WES -1	1-11-77	6	18 of 18	84.0 - 89.2	402.7 - 397.5		
CH1-11 CON-2(A)	MD WES-2	1-11-77	6	1 of 16	0.0 - 4.6	486.7 - 482.1	486.7	
(B)				2 of 16	27.9 - 30.9	458.8 - 455.8		
CH1-11 CON-2(C)				3 of 16	30.9 - 35.5	455.8 - 451.2		
CH1-11 DC-2(A)				4 of 16	35.5 - 39.1	451.2 - 447.6		
(B)				5 of 16	38.1 - 44.2	447.6 - 442.5		
(C)				6 of 16	44.2 - 49.0	442.5 - 437.7		
(D)				7 of 16	49.0 - 52.4	437.7 - 434.3		
(E)				8 of 16	52.4 - 57.0	434.3 - 429.7		
(F)				9 of 16	57.0 - 61.6	429.7 - 425.1		
(G)				10 of 16	61.6 - 65.0	425.1 - 421.7		
(H)				11 of 16	65.0 - 69.7	421.7 - 417.0		
(I)				12 of 16	69.7 - 72.5	417.0 - 414.2		
(J)				13 of 16	72.5 - 76.5	414.2 - 410.2		
(K)				14 of 16	76.5 - 80.8	410.2 - 405.9		
(L)				15 of 16	80.8 - 83.9	405.9 - 402.8		
CH1-11 DC-2(M)	MD WES-2	1-11-77	6	16 of 16	83.9 - 88.4	402.8 - 398.3		
CH1-11 CON-3(A)	MD WES-3	1-11-77	6	1 of 17	0.0 - 4.1	486.7 - 482.6	486.7	
(B)				2 of 17	13.1 - 17.5	473.6 - 469.2		
(C)				3 of 17	30.0 - 34.5	456.7 - 452.2		
CH1-11 DC-3(A)				4 of 17	34.5 - 37.4	452.2 - 449.3		
(B)				5 of 17	37.4 - 39.7	449.3 - 447.0		
(C)				6 of 17	39.7 - 44.0	447.0 - 442.7		
(D)				7 of 17	44.0 - 47.8	442.7 - 438.9		
(E)				8 of 17	47.8 - 42.2	438.9 - 434.5		
(F)				9 of 17	52.2 - 56.3	434.5 - 430.4		
(G)				10 of 17	56.3 - 60.1	430.4 - 426.6		
(H)				11 of 17	60.1 - 64.5	426.6 - 422.2		
(I)				12 of 17	64.5 - 68.2	422.2 - 418.5		
(J)				13 of 17	68.2 - 73.8	418.5 - 412.9		
(K)				14 of 17	73.8 - 76.1	412.9 - 410.6		
(L)				15 of 17	76.1 - 80.5	410.6 - 406.2		
(M)				16 of 17	80.5 - 82.1	406.2 - 404.6		
CH1-11 DC-3(N)	MD WES-3	1-11-77	6	17 of 17	82.1 - 85.9	404.6 - 400.8		
CH1-11 DC-4(A)	MD WES-4	1-11-77	6	1 of 12	24.6 - 29.0	463.6 - 459.2	488.2	El top of casing 488.2
(B)				2 of 12	29.0 - 34.0	459.0 - 454.2		El top of water 483.6
(C)				3 of 12	34.0 - 38.4	454.2 - 449.8		El top of rock 463.6
(D)				4 of 12	38.4 - 43.0	449.8 - 445.2		
(E)				5 of 12	43.0 - 47.6	445.2 - 440.6		
(F)				6 of 12	47.6 - 51.9	440.6 - 436.3		
(G)				7 of 12	51.9 - 55.9	436.3 - 432.3		
(H)				8 of 12	55.9 - 60.5	432.3 - 427.7		
(I)				9 of 12	60.5 - 66.0	427.7 - 422.2		
(J)				10 of 12	66.0 - 69.5	422.2 - 418.7		
(K)				11 of 12	69.5 - 77.5	418.7 - 410.7		
CH1-11 DC-4(L)	MD WES-4	1-11-77	6	12 of 12	77.5 - 83.9	410.7 - 404.3		
CH1-11 DC-5(A)	MD WES-5	1-11-77	6	1 of 16	0.0 - 4.0	469.0 - 465.0	469.0	Concrete from El 469.0 - 467.35
(B)				2 of 16	4.0 - 9.2	465.0 - 459.8		
(C)				3 of 16	9.2 - 13.4	459.8 - 455.6		
(D)				4 of 16	13.4 - 17.9	455.6 - 451.1		
(E)				5 of 16	17.9 - 22.8	451.1 - 446.2		
(F)				6 of 16	22.8 - 27.3	446.2 - 441.7		
(G)				7 of 16	27.3 - 32.2	441.7 - 436.8		
(H)				8 of 16	32.2 - 36.2	436.8 - 432.8		
(I)				9 of 16	36.2 - 40.4	432.8 - 428.6		
(J)				10 of 16	40.4 - 44.1	428.6 - 424.9		
(K)				11 of 16	44.1 - 47.5	424.9 - 421.5		
(L)				12 of 16	47.5 - 51.6	421.5 - 417.4		
(M)				13 of 16	51.6 - 56.0	417.4 - 413.0		
(N)				14 of 16	56.0 - 61.6	413.0 - 407.4		
(O)				15 of 16	61.6 - 65.4	407.4 - 403.6		
CG1-11 DC-5(P)	MD WES-5	5-10-77	6	16 of 16	65.4 - 70.0	403.6 - 399.0		

Table 3

Test Results of Concrete Cores

Marseilles Dam

		Characterization Test						Engineering Design Test	
Drill Hole No.	Elevation ft msl	Effective Unit Wt γ_m lb/ft^3	Dry Unit Wt γ_d, lb/ft^3	Water Content w, %	Comp Wave Velocity V_p, fps	Comp Strength UC, psi	Tensile Strength T_s, psi	Elastic Modulus Ex 10^6 psi	Poisson's Ratio
1	484.4	153.6	144.9	6.0	16,595	8,090		4.26	0.18
1	471.5	154.8	145.9	6.1	17,240	8,250		5.36	0.21
1	462.3	155.4	147.7	5.2	16,950	10,020		5.01	0.19
1	454.3			6.1			680		
2	484.5	154.8	146.7	5.5	16,880	8,860		4.78	0.19
2	483.5			5.6			630		
2	458.2	156.1	147.8	5.6	17,100	10,280		5.52	0.20
2	455.2	155.5	148.8	4.5	17,645	9,600		5.55	0.19
3	483.1	154.8	145.5	6.4	17,170	9,350		6.07	0.19
3	471.2			5.7			665		
3	470.2	154.8	145.8	6.2	17,400	8,830		4.97	0.19
3	452.7	156.7	149.8	4.6	17,900	10,090		5.11	0.27

Table 4

Characterization Properties

Rock Cores, Marseilles Dam

Drill Hole No.	Elevation ft msl	Effective Unit Wt γ_m, lb/ft^3	Dry Unit Wt γ_d, lb/ft^3	Water Content $\bar{w}$, %	Comp Wave Velocity V_p, fps	Comp Strength UC, psi	Tensile Splitting Strength T_s, psi	Remarks
1	452.4			7.6			90	Shale, Zone 1*
1	450.3	158.2	150.6	5.0	8,020	2,100		Shale, Zone 1
1	446.7	158.3	149.0	6.0	7,960	2,360		Shale, Zone 1
1	437.9	153.1	146.0	4.8	6,995	340		Shale, Zone 2
1	436.9	152.3	144.4	5.5	6,930	380		Shale, Zone 2
1	432.7	150.0	136.1	10.2	6,525	330		Shale, Zone 2
1	420.1	137.1						Claystone
1	419.2	149.3						Claystone
1	413.7			7.0		20		Claystone
1	409.2	165.8	165.1	0.4	16,995	6,650		Sandstone
1	407.6	165.3	164.3	0.6	18,690	14,140		Sandstone
2	451.7			6.4		1,730		Shale, Zone 1
2	446.0			6.7			110	Shale, Zone 1
2	444.1			7.3		1,920		Shale, Zone 1
2	432.1	146.3	135.5	7.9		330		Shale, Zone 2
2	422.5			11.7		50		Claystone
2	401.2	148.9	143.0	4.1	8,425	4,740		Sandstone
2	400.1	146.5	142.0	3.2	7,790	3,310		Sandstone
3	448.8	160.3	151.8	5.6	9,175	2,280		Shale, Zone 1
3	443.7			7.5		2,190		Shale, Zone 1
3	442.9			6.2				Shale, Zone 1
3	441.4			5.9			140	Shale, Zone 1
3	432.4	142.6	131.5	8.4		310		Shale, Zone 2
3	422.8	94.3	84.0	12.3		2,570		Coal
3	416.2	136.0	121.9	11.6		90		Claystone
3	410.2					80		Claystone
4	449.3					2,510		Shale, Zone 1
4	447.7						145	Shale, Zone 1
4	438.5					860		Shale, Zone 2
5	442.6					2,190		Shale, Zone 1

* Zone 1 shale is more competent while Zone 2 shale is more clayey. Zone 1 is about 12 ft thick and occurs just below the pier key. Zone 2 is about 14 ft thick and occurs just below Zone 1.

Table 5

Engineering Design Tests

Rock Cores, Marseilles Dam

Drill Hole No.	Elevation ft msl	Elastic Modulus $E \times 10^6$ psi	Poisson's Ratio	Triaxial σ_3, psi	Triaxial σ_1, psi	Pullout Resistance psi	Remarks
1	450.3	0.18	0.45				Shale, Zone 1
1	446.7	0.17	0.47				Shale, Zone 1
1	437.9	0.04	0.44				Shale, Zone 2
1	436.9	0.05	0.24				Shale, Zone 2
1	432.7	0.04	0.36				Shale, Zone 2
2	451.7	0.11*					Shale, Zone 1
2	449.8					192.95	Shale, Zone 1
2	444.1	0.12	--				Shale, Zone 1
2	440.7			27	1206		
2	439.7			55	1286		
2	438.7			111	2086		
2	432.1	0.01	--				Shale, Zone 2
2	422.5	0.01					Claystone
2	429.2					28.87	Shale, Zone 2
3	448.8	0.14	0.47				Shale, Zone 1
3	443.7	0.13	--				Shale, Zone 1
3	432.4	0.01	--				Shale, Zone 1
3	431.1					32.89	Shale, Zone 2
3	429.1					32.46	Shale, Zone 2
3	422.8	0.4	--				Coal
3	416.2	0.01	--				Claystone
3	410.2	0.02	--				Claystone
4	449.3	0.17	--				Shale, Zone 1
4	442.9					187.65	Shale, Zone 1
4	441.8					267.40	Shale, Zone 1
4	438.5	0.03	--				Shale, Zone 2
5	442.6	0.13	--				Shale, Zone 1

* Specimens tested in axial deformation rig, no radial measurements were taken, therefore no Poisson's ratio obtained.

Table 6

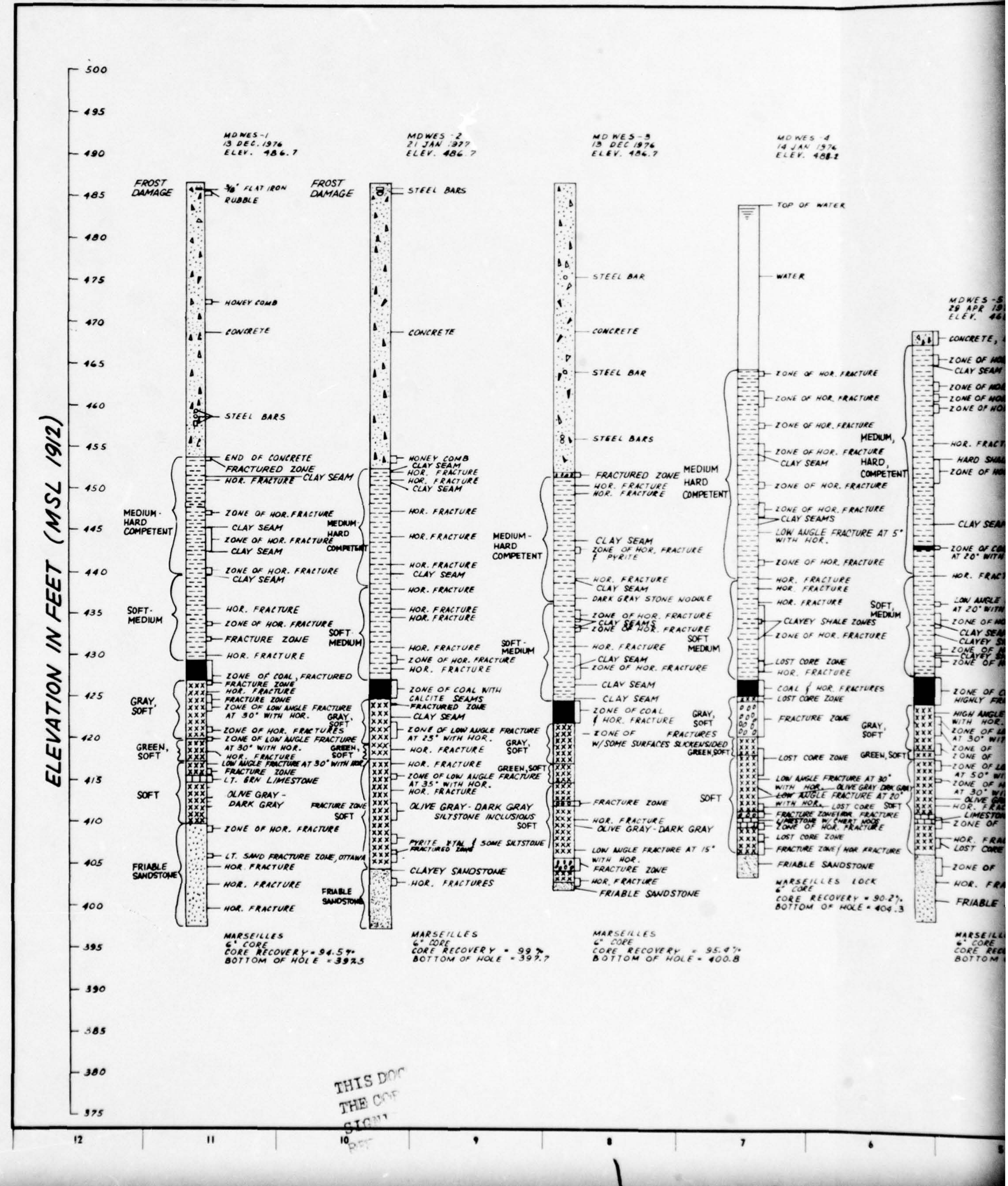
Laboratory Test Results - Marseilles Dam

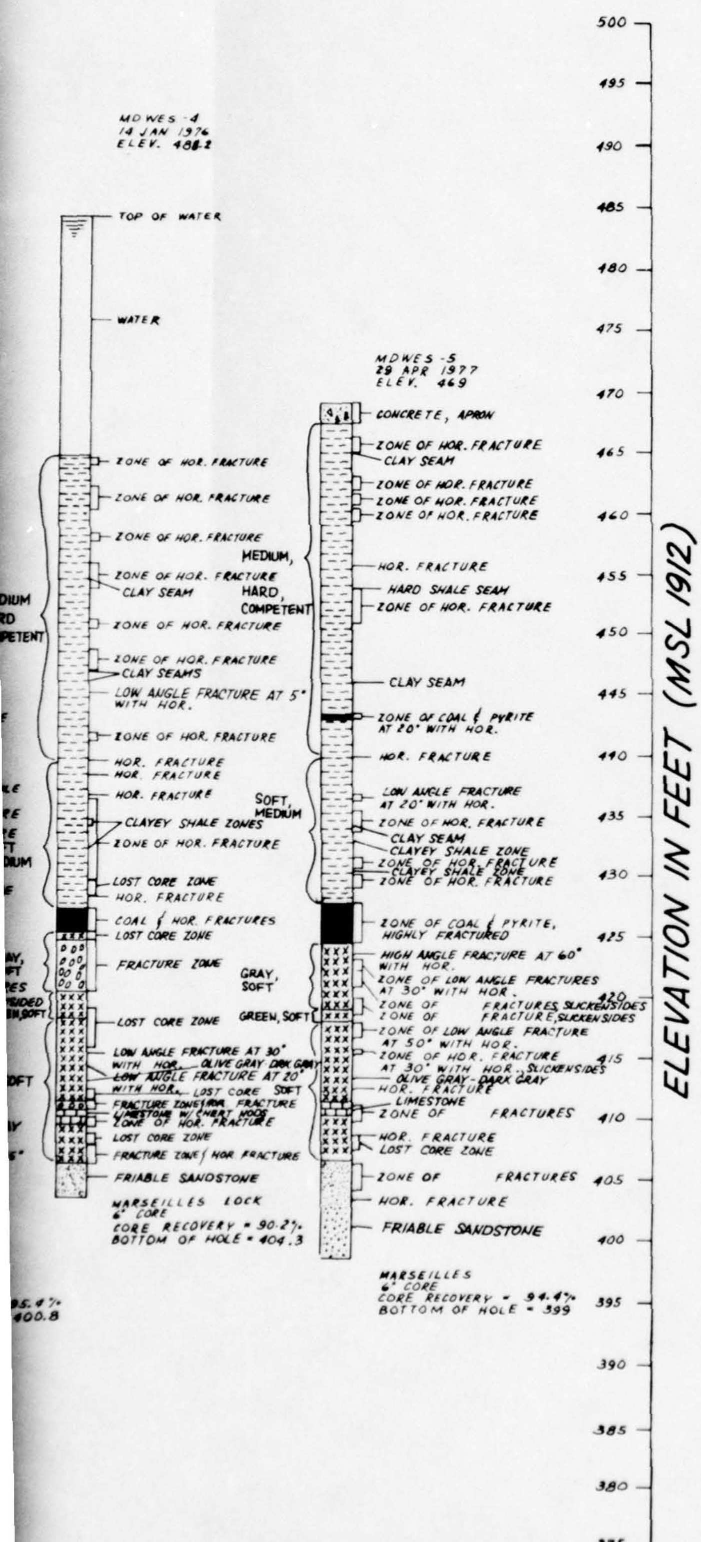
Single-Plane Shear Tests

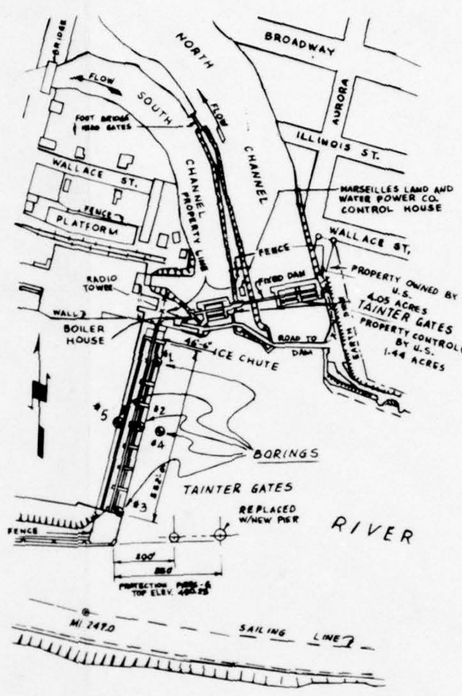
Rock Type	Drill Hole No. MD WES	Elev, ft	Type Test	Normal Stress tsf	Peak Shear Stress, tsf	Peak Shear Strength
Shale	1-76	453.3	Concrete to rock	2	3.59	C = 1.06 tsf
	2-77	452.7		4	8.76	$\phi = 67^o$
	3-76	451.2		8	18.06	
Shale	1-76	448.4	Intact*	4	9.19	C = 5.19 tsf
	1-76	447.4		8	12.89	$\phi = 45^o$
	1-76	444.5		8	13.50	
Shale	2-77	450.5	Intact*	2	2.90	C = 1.03 tsf
	2-77	449.5		4	5.70	$\phi = 47^o$
	2-77	448.7		8	9.45	
Clay Seam in shale (seam <2 mm)	3-76	444.4	Intact	2	3.72	C = 2.69 tsf
	4-77	454.3		4	3.25	$\phi = 17^o$
	4-77	446.0		8	5.32	
Clay Seam in shale (seam <2 mm)	5-77	448.5	Intact	2	9.21	C = 7.81 tsf
		448.1		4	8.09	$\phi = 19^o$
		447.7		8	10.09	
Shale	2-77	450.0	Precut, rock on rock	2	4.74	C = 3.33 tsf
				4	4.75	$\phi = 27^o$
				8	7.57	
Shale	4-77	445.7	Natural joint	2	2.00	C = 0.64 tsf
				4	4.16	$\phi = 38^o$
				8	6.88	
Shale	2-77	442.3	Open bedding plane	2	1.75	C = 0.68 tsf
				4	2.28	$\phi = 25^o$
				8	4.43	
Shale	3-76	450.3	Cross bed	2	4.57	C = 2.63 tsf
	2-77	445.1		4	5.97	$\phi = 42^o$
	1-76	443.5		8	9.85	

*Parallel to bedding.

CORPS OF ENGINEERS







LOCATION OF BORINGS

GRAPHIC SYMBOLS

FRACTURE ZONE

LIMESTONE

SHALE - DARK GRAY; MEDIUM HARD; OCCASIONAL THIN CLAY SEAMS < 2 MM., FISSIBLE; BOTTOM 12 TO 13 IS MORE CLAYEY; 23 TO 28 FT. THICK

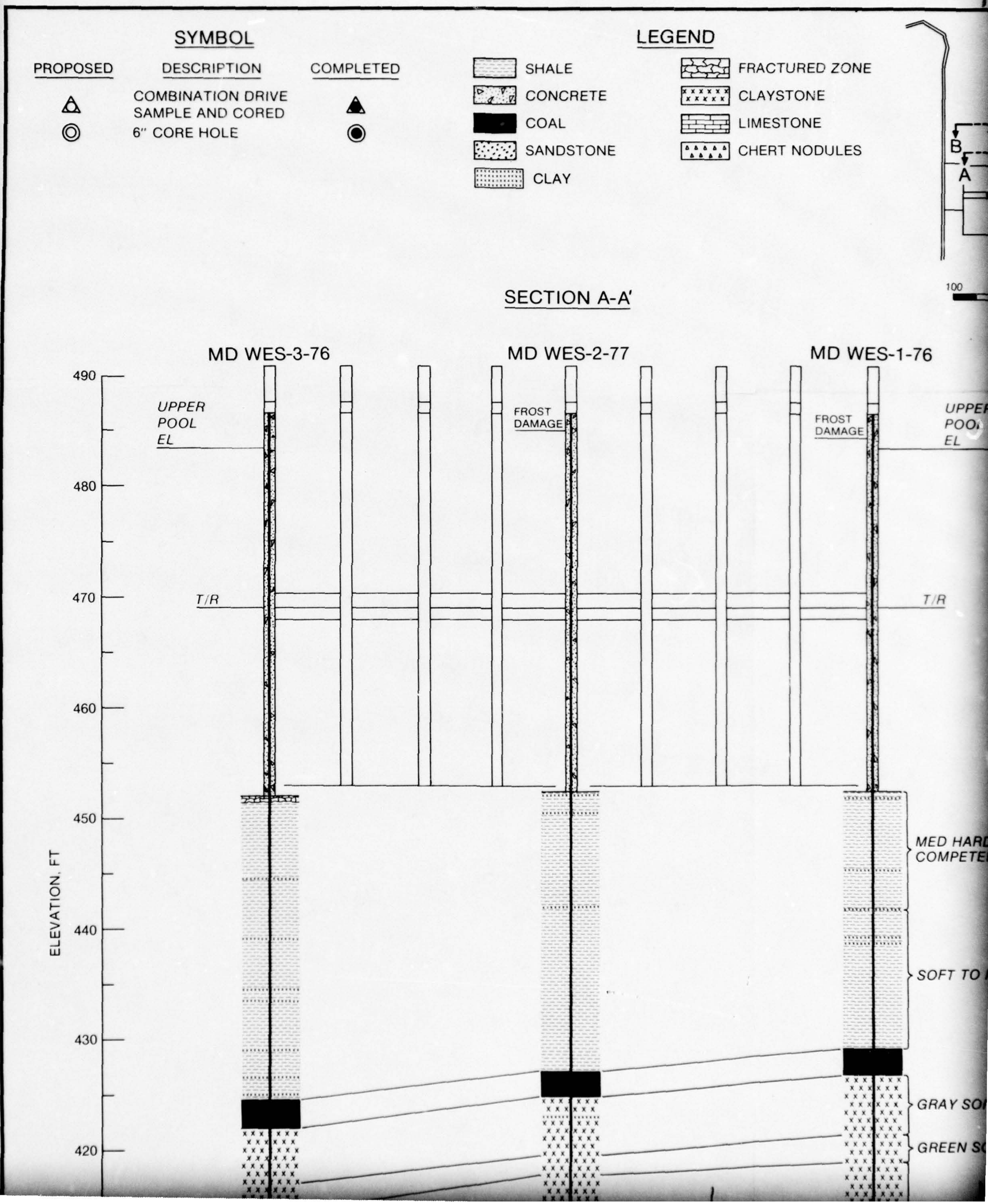
COAL - BLACK; HIGHLY FRACTURED, BLOCKY, SOME FRACTURES FILLED WITH CALCITE; 2 TO 2.5 FT. THICK

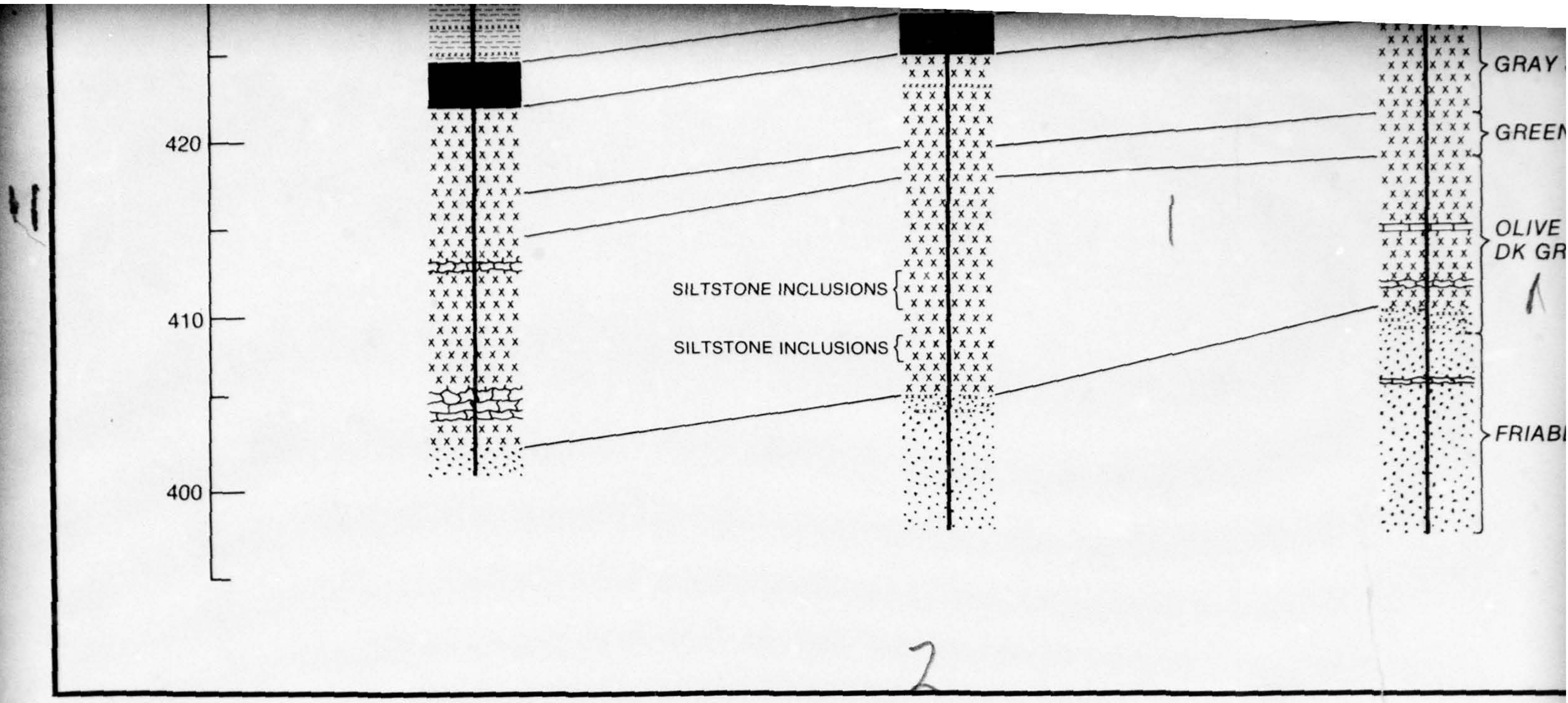
GRAY — CLAYSTONE - DARK GRAY TO BROWNISH BLACK, CLAYEY, SOFT, AND CRUMBLY; HIGHLY FRACTURED WITH AT LEAST 3 JOINT SETS, SLICKENSIDED SURFACES PRODUCED BY POSSIBLE CONTEMPORANEOUS DEFORMATION; 5.5 FT. THICK.

GREEN — CLAYSTONE - DUSKY YELLOW-GREEN; SOFT AND CRUMBLY; HIGHLY FRACTURED; JOINT SETS, SLICKENSIDED; 1 TO 2 FT. THICK.

CLAYSTONE - OLIVE GRAY TO DARK GRAY; SOME ORGANIC DEBRIS, CONTAINS A 1-FT. THICK BED OF GREEN CLAYSTONE NOT CONTINUOUS BETWEEN BORINGS; STONE NODULES AND PYRITE CRYSTALS THROUGHOUT; DISCONTINUOUS LIMESTONE BED IN BORING 1 TO 5; BORING 4 CONTAINS A 1-FT. BED OF CONGLOMERATE WITH CHERT NODULES; VERY HARD LIMESTONE TO SOFT HIGHLY FRACTURED CLAYSTONE, 10 TO 13 FT. THICK

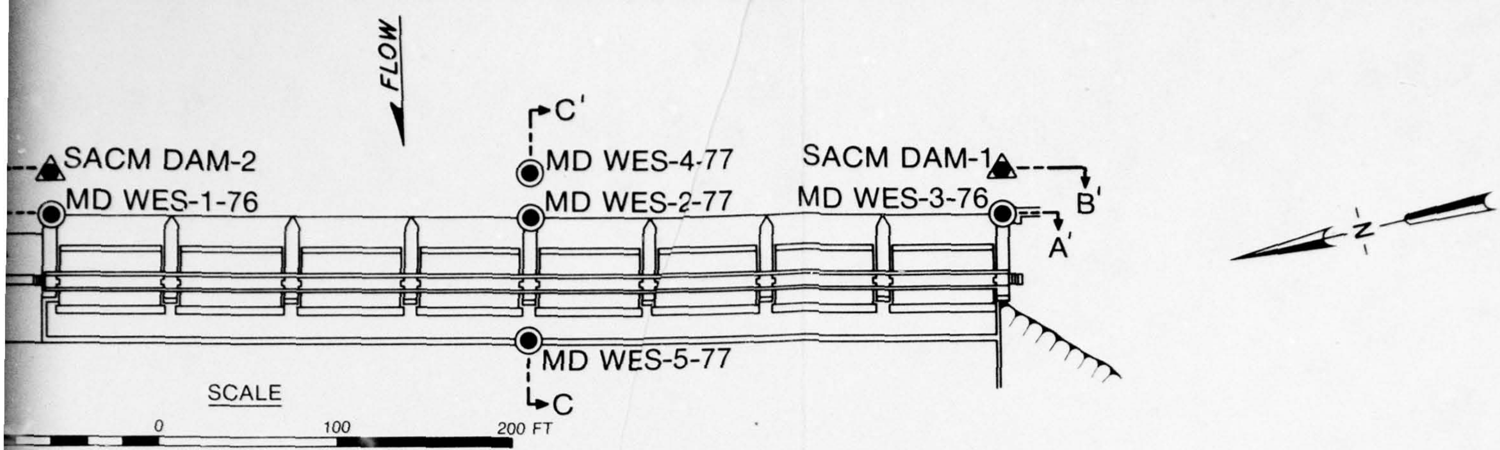
SANDSTONE - LIGHT GRAY; MEDIUM-GRAINED, VERY FRIABLE, LOOSELY CEMENTED, TOP FOOT GRADES FROM CLAYEY SANDSTONE TO SANDSTONE; THIRTEEN FEET THICK IN BORING NO. 1.


SYMBOL
PROPOSED
DESCRIPTION
COMPLETED
COMBINATION DRIVE SAMPLE AND CORED
6" CORE HOLE
LEGEND
SHALE
CONCRETE
COAL
SANDSTONE
CLAY
FRACTURED ZONE
CLAYSTONE
LIMESTONE
CHERT NODULES
B
A
100
SECTION A-A'
MD WES-3-76
MD WES-2-77
MD WES-1-76
490
480
470
460
450
440
430
420
ELEVATION, FT
UPPER POOL EL
FROST DAMAGE
FROST DAMAGE
T/R
T/R
MED HARD
COMPETE
SOFT TO
GRAY SO
GREEN S

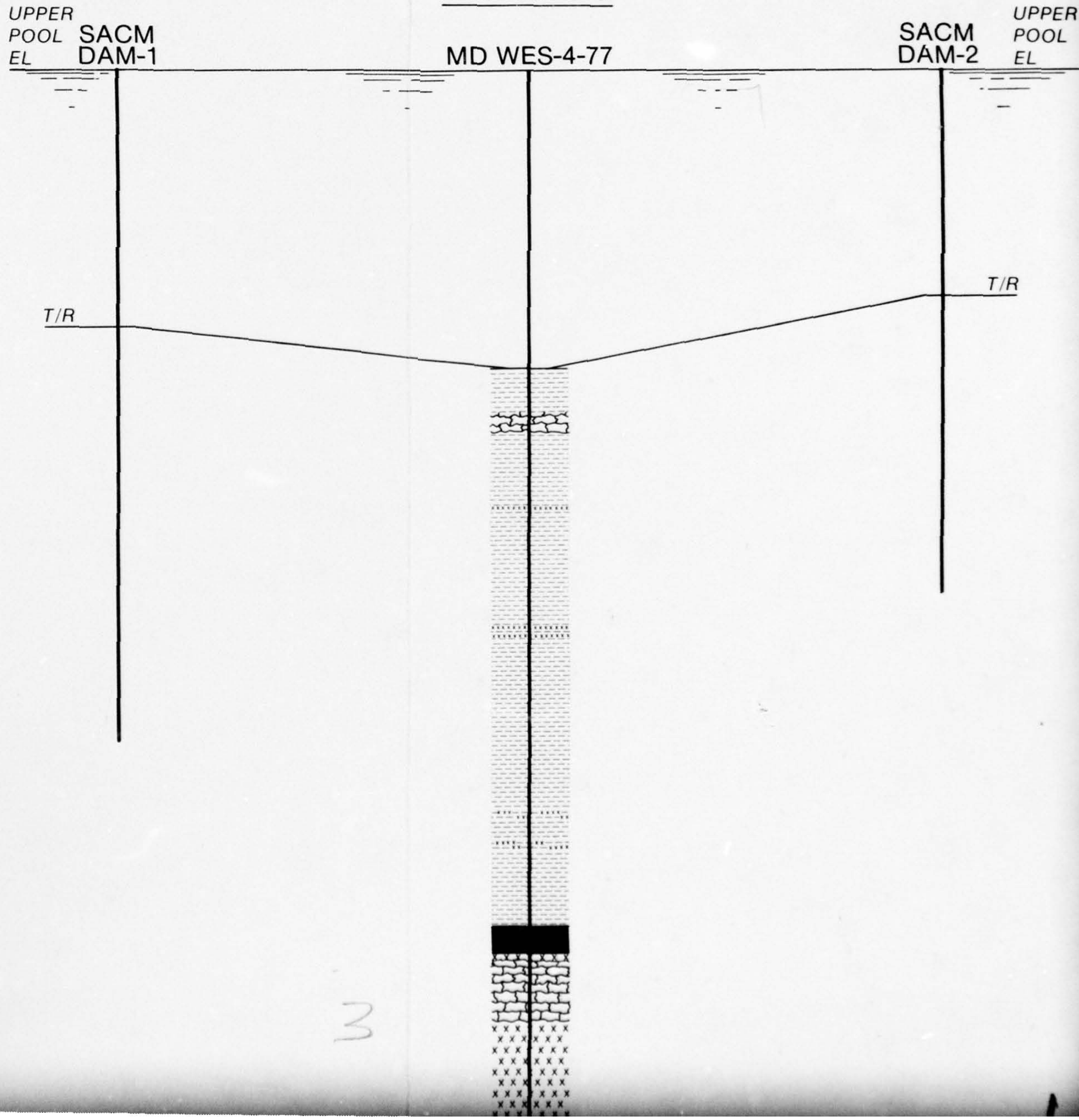


420
410
400
SILTSTONE INCLUSIONS
SILTSTONE INCLUSIONS
GRAY
GREEN
OLIVE
DK GR
FRIAB
1
1
2

3



SECTION B-B'



PER
OL

ARD
TENT

TO MED

SOFT

SOFT

3

OFT

SOFT

GRAY TO
Y SOFT

E

3

STABILIZATION PHASE

MARSEILLES DAM
ILLINOIS WATERWAY

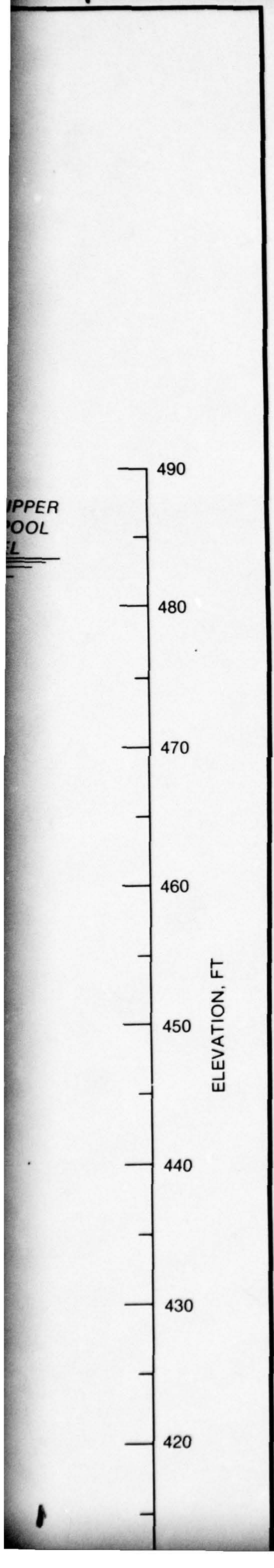
GEOLOGIC CROSS SECT

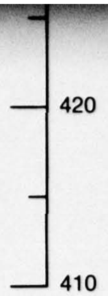
SECTIONS A-A' AND B-B

4

1

5



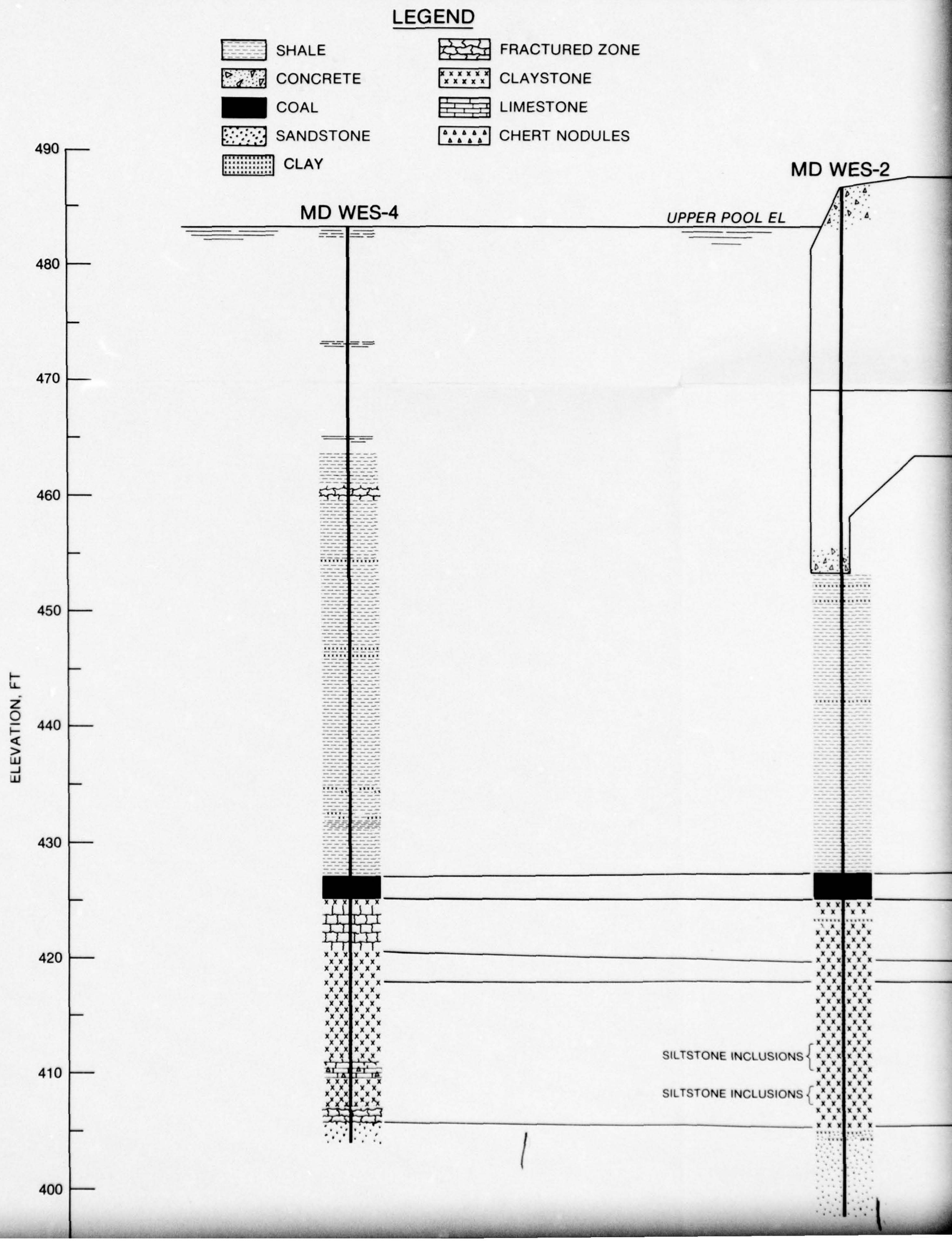


ATION PHASE
ILLES DAM
WATERWAY

ROSS SECTIONS
A-A' AND B-B'

6

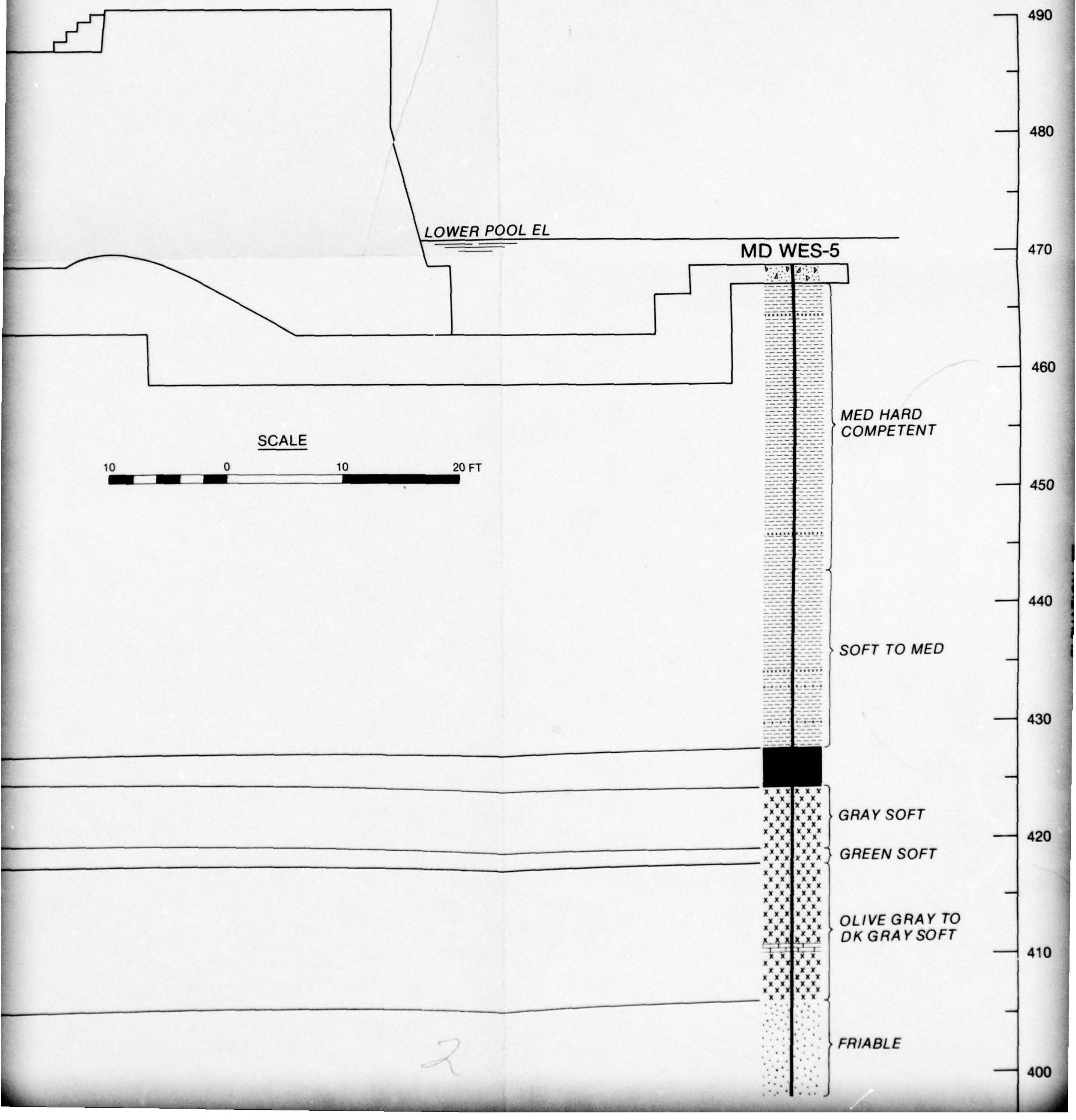
PLATE 2

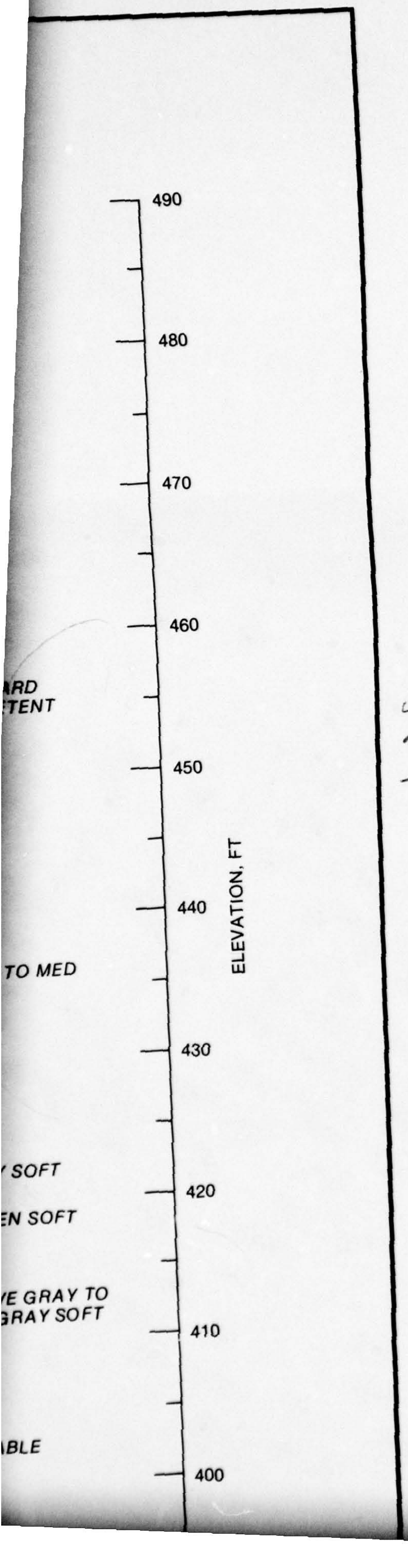


LEGEND
SHALE
CONCRETE
COAL
SANDSTONE
CLAY
FRACTURED ZONE
CLAYSTONE
LIMESTONE
CHERT NODULES
490
480
470
460
450
440
430
420
410
400
ELEVATION, FT
MD WES-4
UPPER POOL EL
MD WES-2
SILTSTONE INCLUSIONS
SILTSTONE INCLUSIONS

2

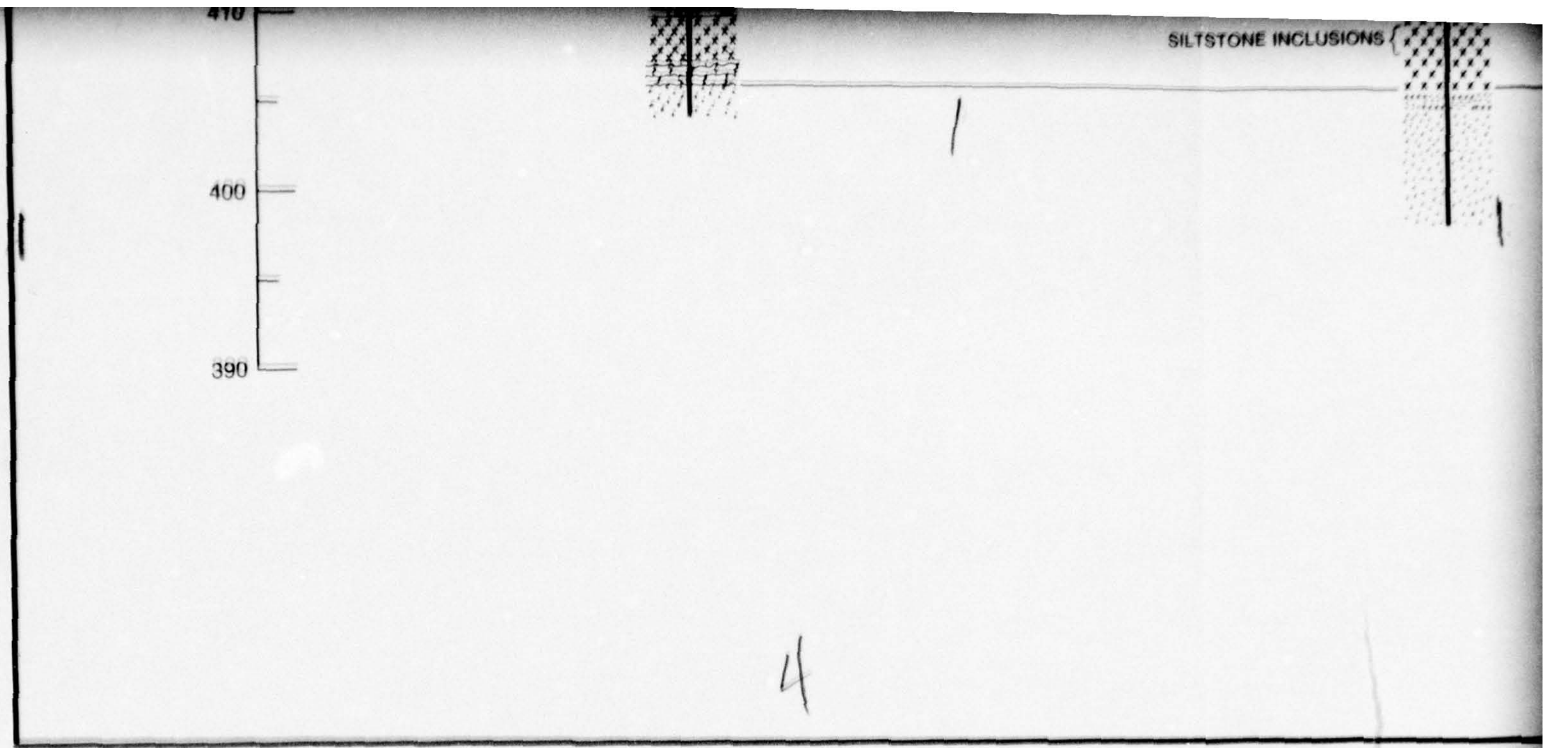
SECTION C-C'

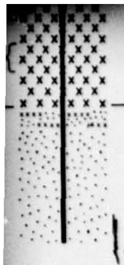

LOWER POOL EL
MD WES-5
MED HARD COMPETENT
SOFT TO MED
GRAY SOFT
GREEN SOFT
OLIVE GRAY TO DK GRAY SOFT
FRIABLE
SCALE
10
0
10
20 FT
490
480
470
460
450
440
430
420
410
400
2



490
480
470
460
ARD
TENT
450
440
ELEVATION, FT
TO MED
430
SOFT
420
EN SOFT
E GRAY TO
GRAY SOFT
410
BLE
400

3


410
400
390
SILTSTONE INCLUSIONS
4



2

STABI

MARS
ILLINO

GEOLOGIC

SEC

5

390

STABILIZATION PHASE

MARSEILLES DAM

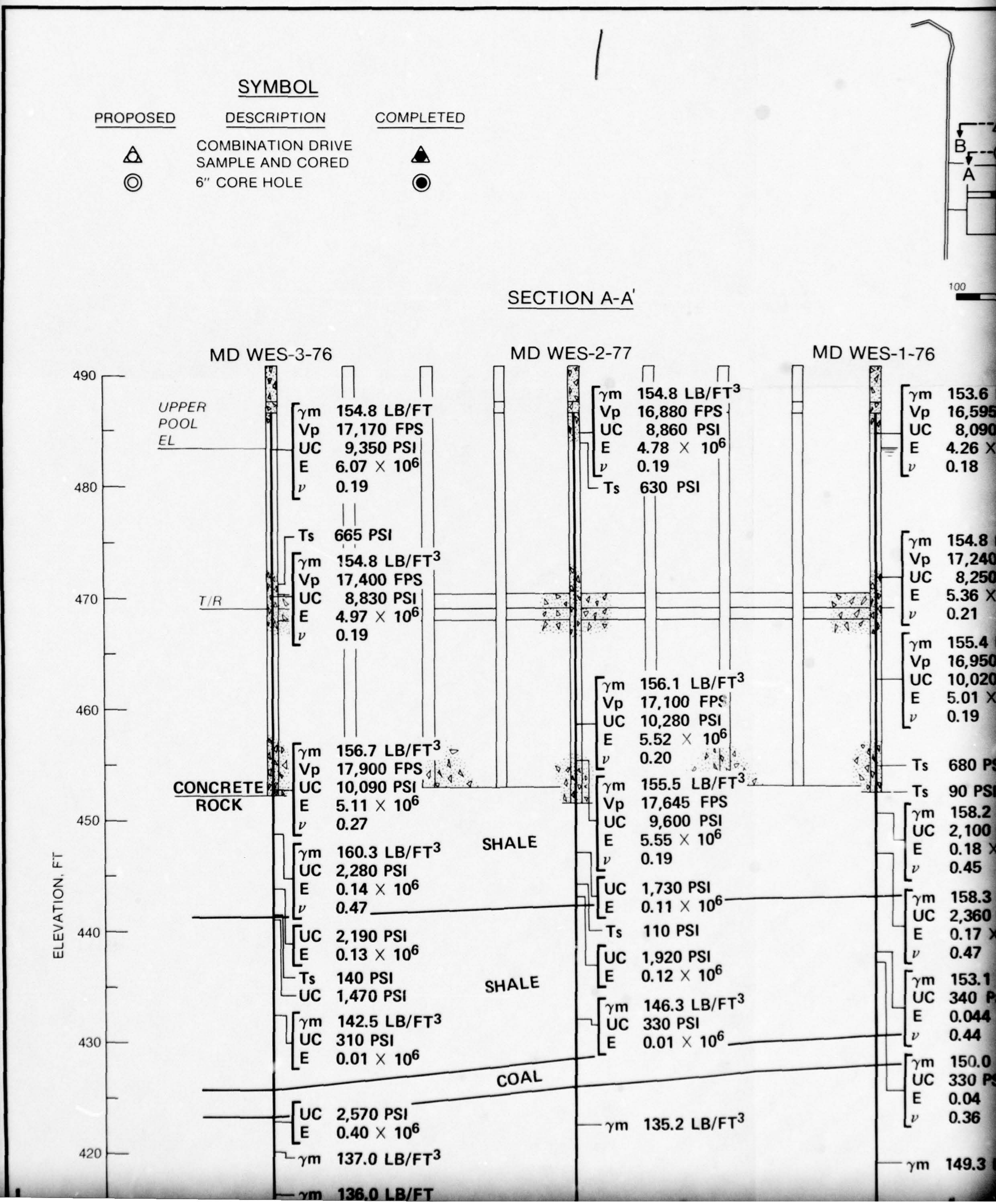
ILLINOIS WATERWAY

LOGIC CROSS SECTION

SECTION C-C'

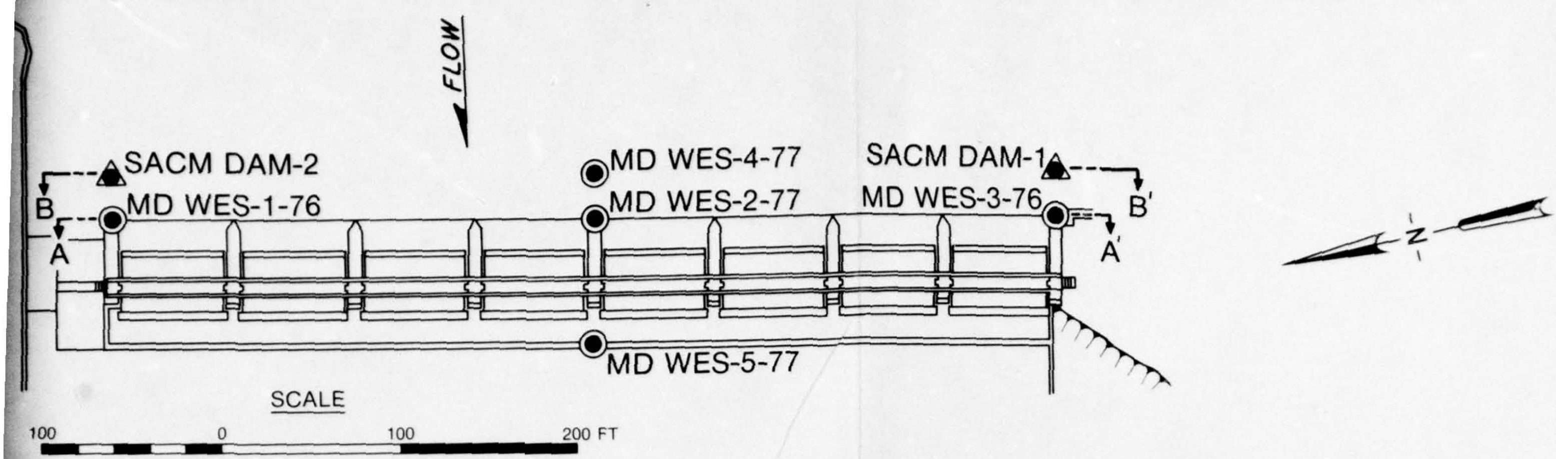
6

PLATE 3

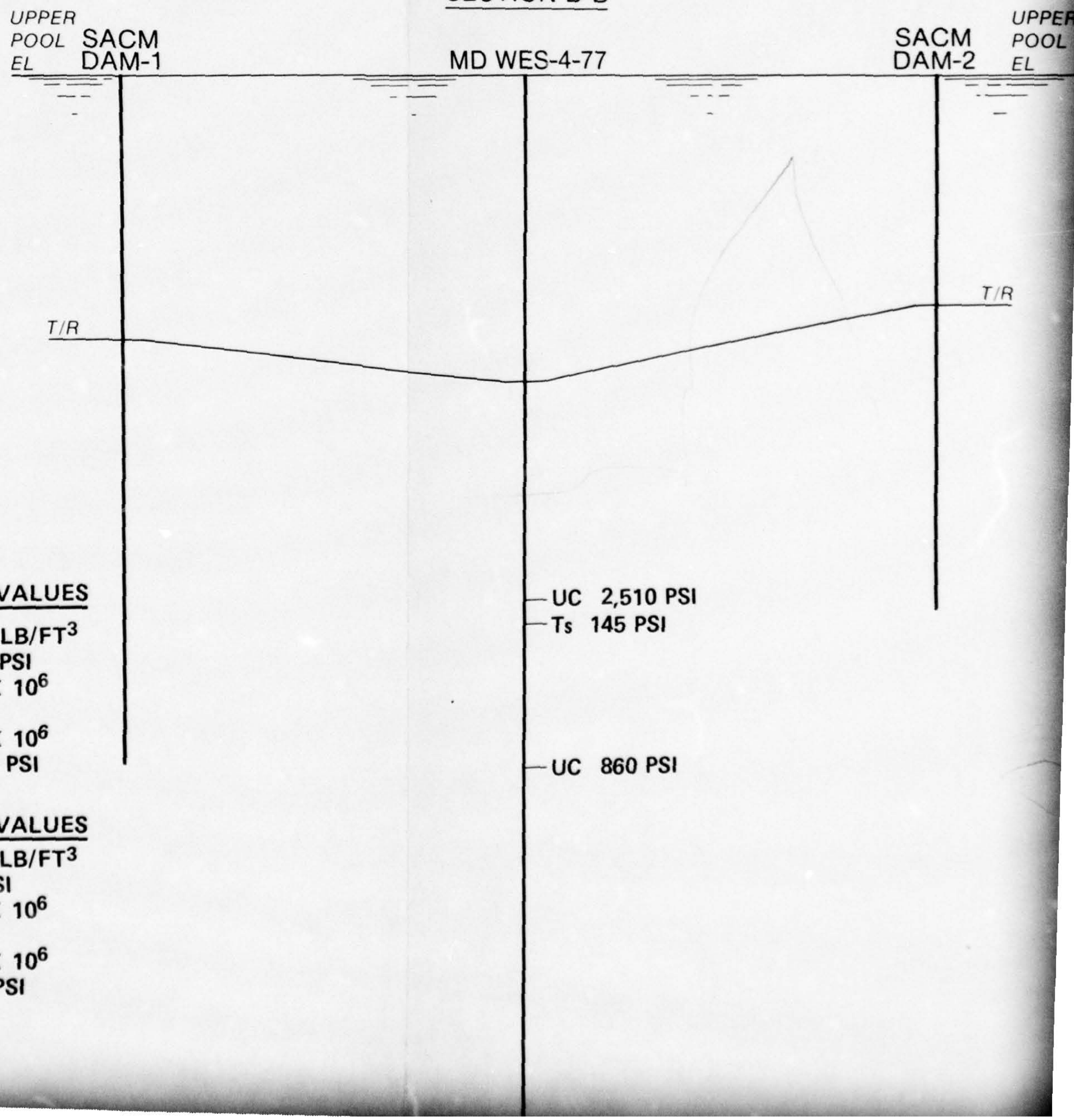


SYMBOL
PROPOSED
DESCRIPTION
COMPLETED
COMBINATION DRIVE SAMPLE AND CORED
6" CORE HOLE
B
A
100
SECTION A-A'
MD WES-3-76
MD WES-2-77
MD WES-1-76
ELEVATION, FT
490
480
470
460
450
440
430
420
UPPER POOL EL
T/R
CONCRETE
ROCK
SHALE
SHALE
COAL
γm 154.8 LB/FT3
Vp 17,170 FPS
UC 9,350 PSI
E 6.07 × 10^6
ν 0.19
Ts 665 PSI
γm 154.8 LB/FT3
Vp 17,400 FPS
UC 8,830 PSI
E 4.97 × 10^6
ν 0.19
γm 156.7 LB/FT3
Vp 17,900 FPS
UC 10,090 PSI
E 5.11 × 10^6
ν 0.27
γm 160.3 LB/FT3
UC 2,280 PSI
E 0.14 × 10^6
ν 0.47
UC 2,190 PSI
E 0.13 × 10^6
Ts 140 PSI
UC 1,470 PSI
γm 142.5 LB/FT3
UC 310 PSI
E 0.01 × 10^6
UC 2,570 PSI
E 0.40 × 10^6
γm 137.0 LB/FT3
γm 136.0 LB/FT
γm 154.8 LB/FT3
Vp 16,880 FPS
UC 8,860 PSI
E 4.78 × 10^6
ν 0.19
Ts 630 PSI
γm 156.1 LB/FT3
Vp 17,100 FPS
UC 10,280 PSI
E 5.52 × 10^6
ν 0.20
γm 155.5 LB/FT3
Vp 17,645 FPS
UC 9,600 PSI
E 5.55 × 10^6
ν 0.19
UC 1,730 PSI
E 0.11 × 10^6
Ts 110 PSI
UC 1,920 PSI
E 0.12 × 10^6
γm 146.3 LB/FT3
UC 330 PSI
E 0.01 × 10^6
γm 135.2 LB/FT3
γm 153.6
Vp 16,595
UC 8,090
E 4.26 ×
ν 0.18
γm 154.8
Vp 17,240
UC 8,250
E 5.36 ×
ν 0.21
γm 155.4
Vp 16,950
UC 10,020
E 5.01 ×
ν 0.19
Ts 680
Ts 90 PSI
γm 158.2
UC 2,100
E 0.18 ×
ν 0.45
γm 158.3
UC 2,360
E 0.17 ×
ν 0.47
γm 153.1
UC 340
E 0.044
ν 0.44
γm 150.0
UC 330
E 0.04
ν 0.36
γm 149.3

1 2



SECTION B-B'



153.6 LB/FT^3
16,595 FPS
8,090 PSI
4.26×10^6
0.18

154.8 LB/FT
17,240 FPS
8,250 PSI
5.36×10^6
0.21

155.4 LB/FT^3
16,950 FPS
10,020 PSI
5.01×10^6
0.19

680 PSI

90 PSI

158.2 LB/FT^3
2,100 PSI
0.18×10^6
0.45

158.3 LB/FT^3
2,360 PSI
0.17×10^6
0.47

AVERAGE VALUES

γm 158.7 LB/FT^3
UC 2,160 PSI
E 0.14×10^6
ν 0.46
G 0.05×10^6
BOND 216 PSI

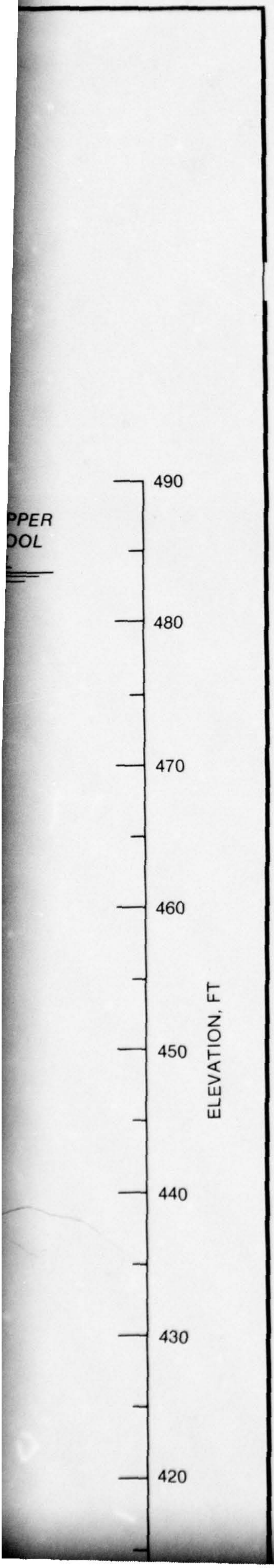
153.1 LB/FT^3
340 PSI
0.044×10^6
0.44

150.0 LB/FT^3
330 PSI
0.04
0.36

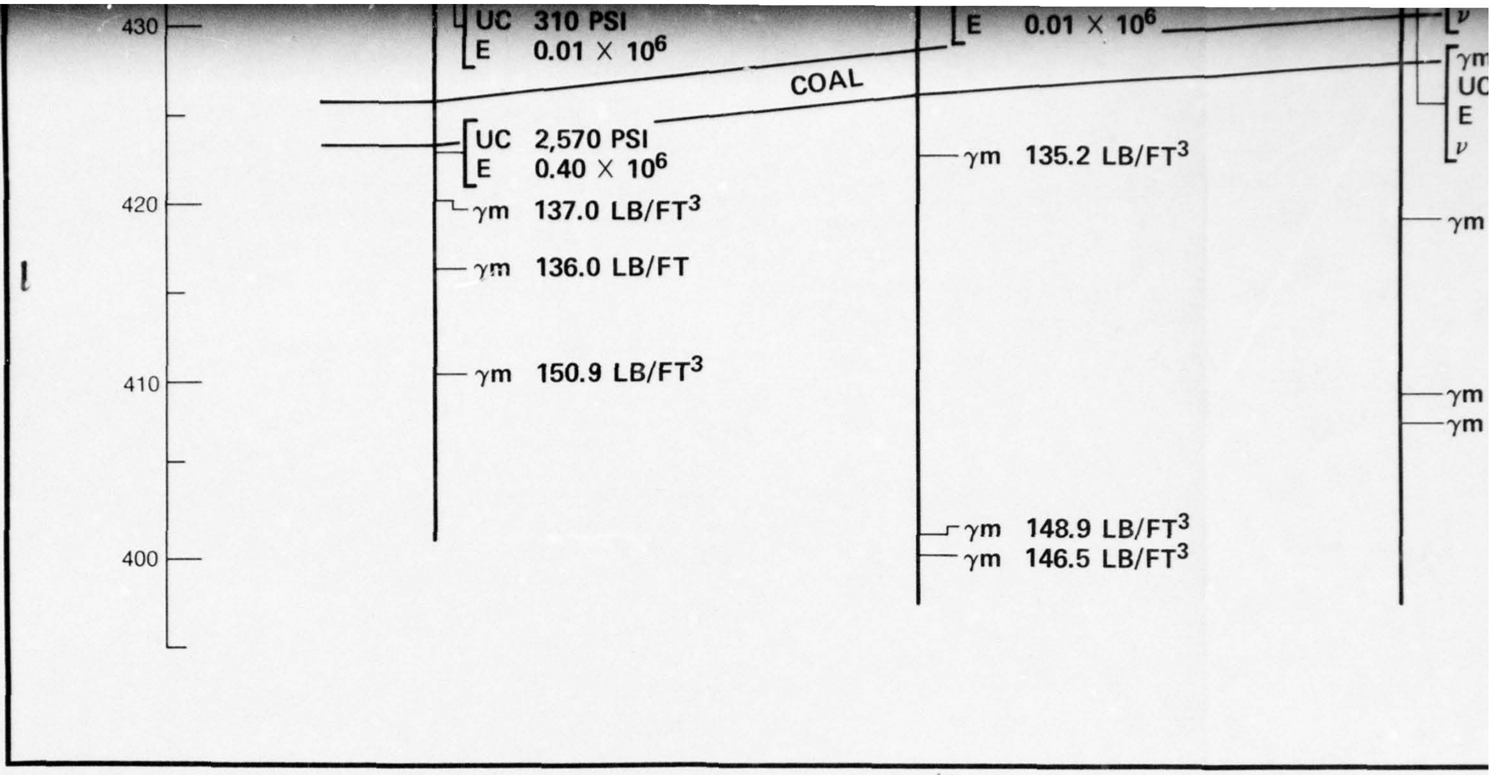
AVERAGE VALUES

γm 148.8 LB/FT^3
UC 430 PSI
E 0.03×10^6
ν 0.35
G 0.01×10^6
BOND 31 PSI

149.3 LB/FT^3



3


430
UC 310 PSI
E 0.01 × 10⁶
COAL
UC 2,570 PSI
E 0.40 × 10⁶
420
γm 137.0 LB/FT³
γm 136.0 LB/FT
410
γm 150.9 LB/FT³
400
E 0.01 × 10⁶
γm 135.2 LB/FT³
γm 148.9 LB/FT³
γm 146.5 LB/FT³
ν
γm
UC
E
ν
γm
γm
γm

0.044 × 10^6
0.44
m 150.0 LB/FT^3
C 330 PSI
0.04
0.36

γm 148.8 LB/FT^3
UC 430 PSI
E 0.03 × 10^6
ν 0.35
G 0.01 × 10^6
BOND 31 PSI

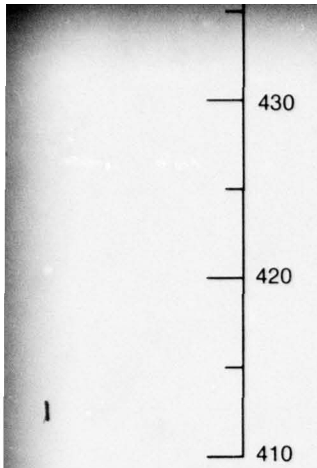
m 149.3 LB/FT^3

m 165.8 LB/FT^3
m 165.3 LB/FT^3

STABILIZATION PHASE
MARSEILLES DAM
ILLINOIS WATERWAY

SELECTED PHYSICAL P
SECTIONS A-A' AND

5

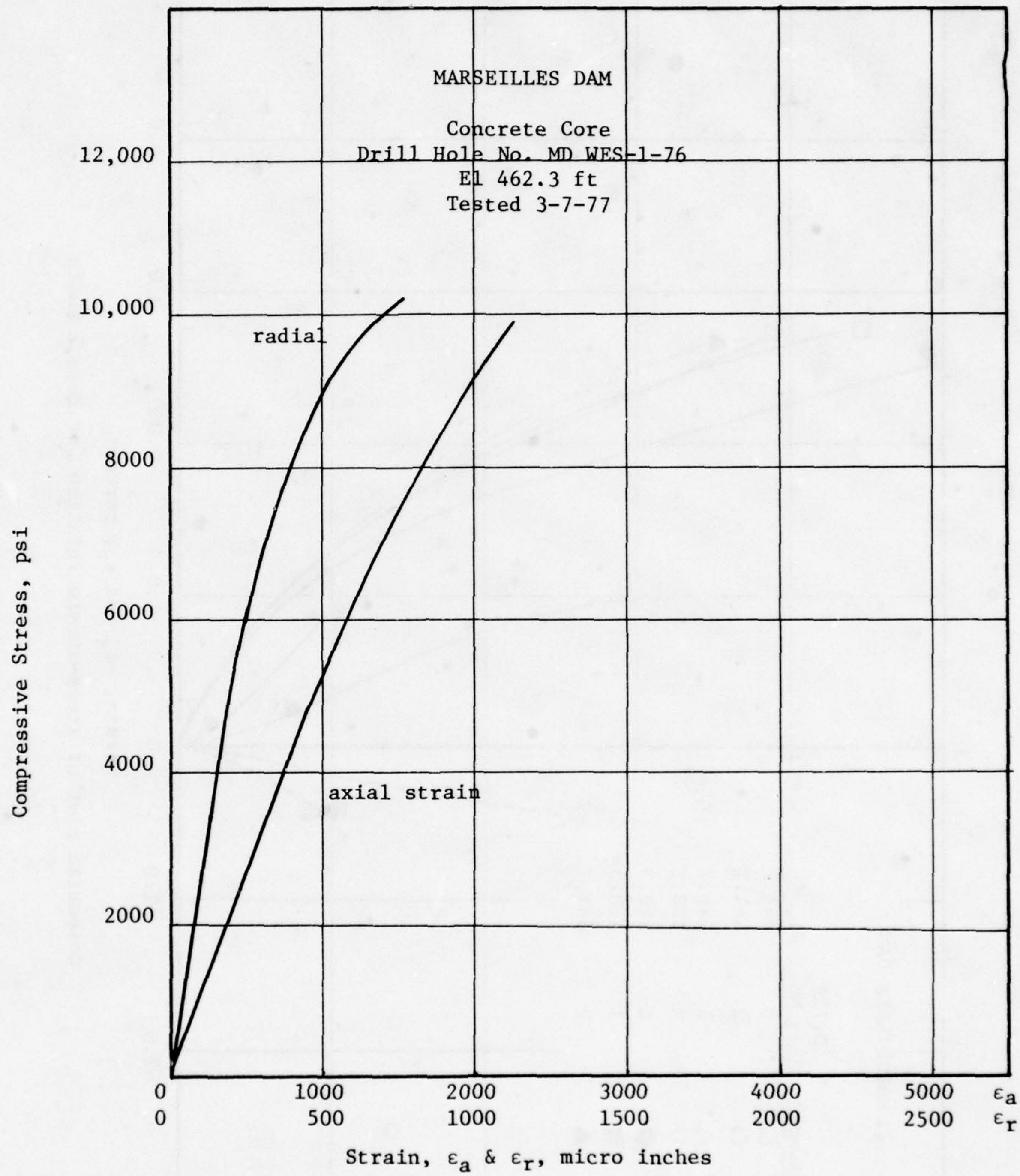


N PHASE
ES DAM
ATERWAY

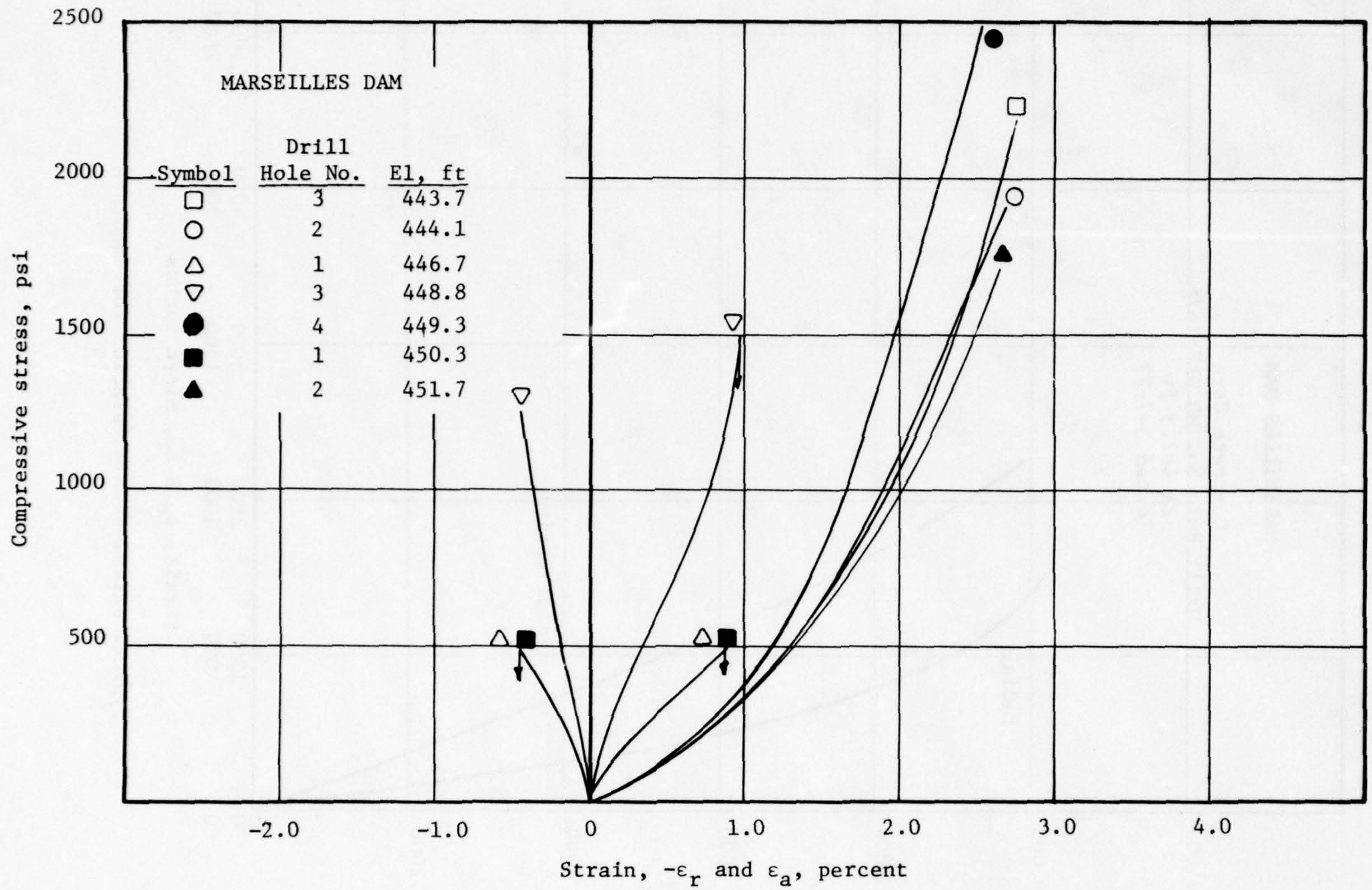
CAL PROPERTIES
A' AND B-B'

6

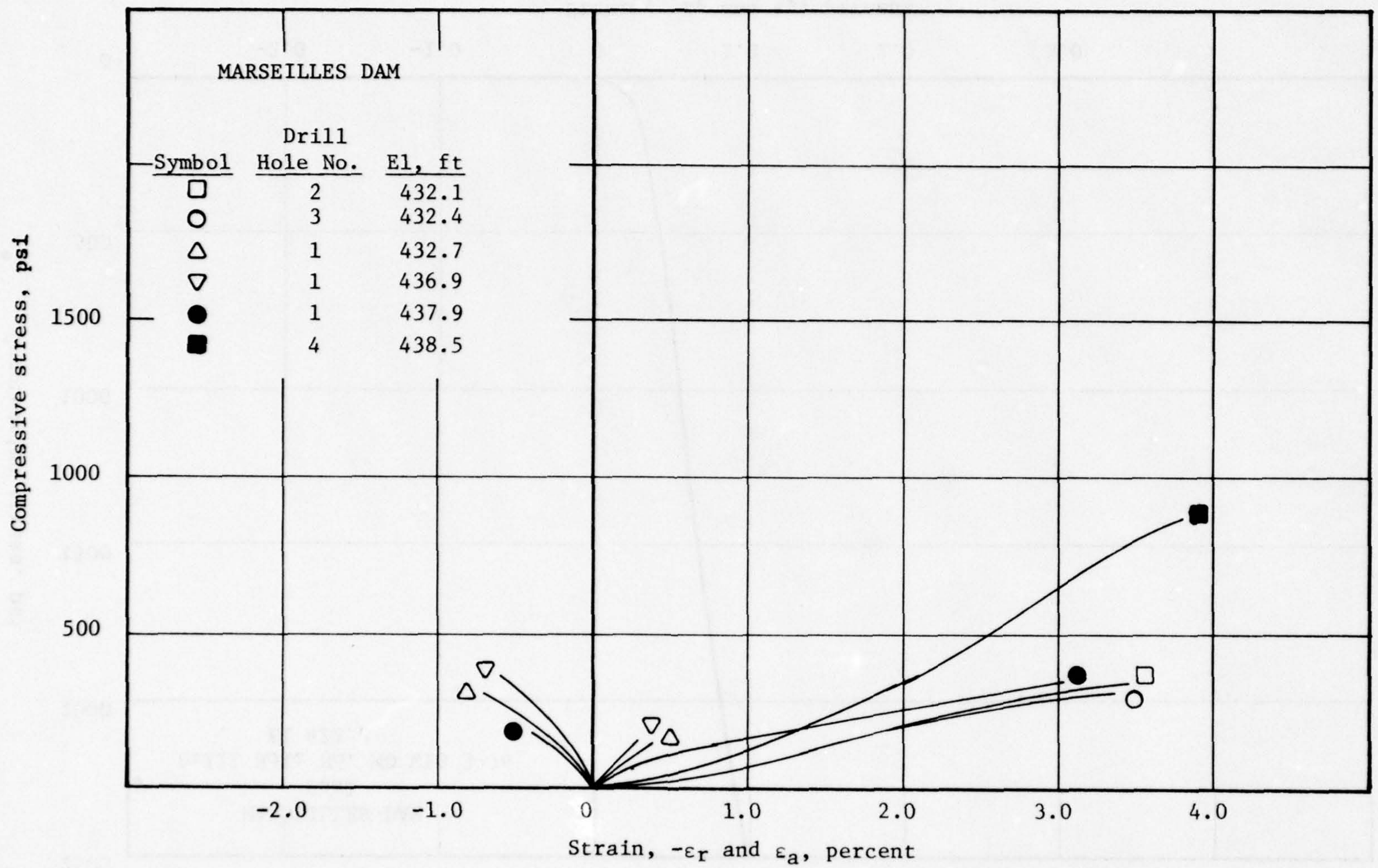
PLATE 4



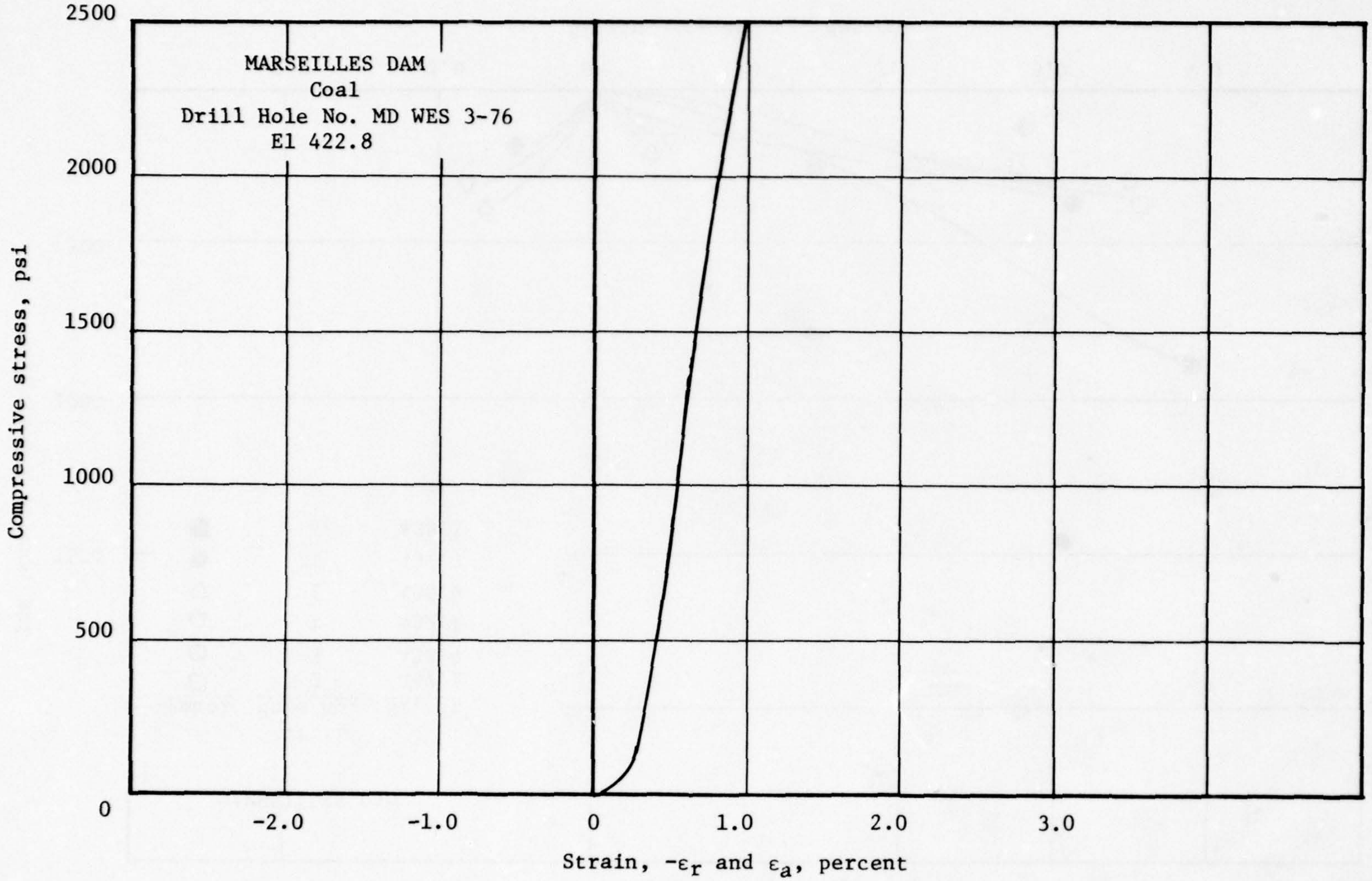
Typical stress-strain relation for concrete



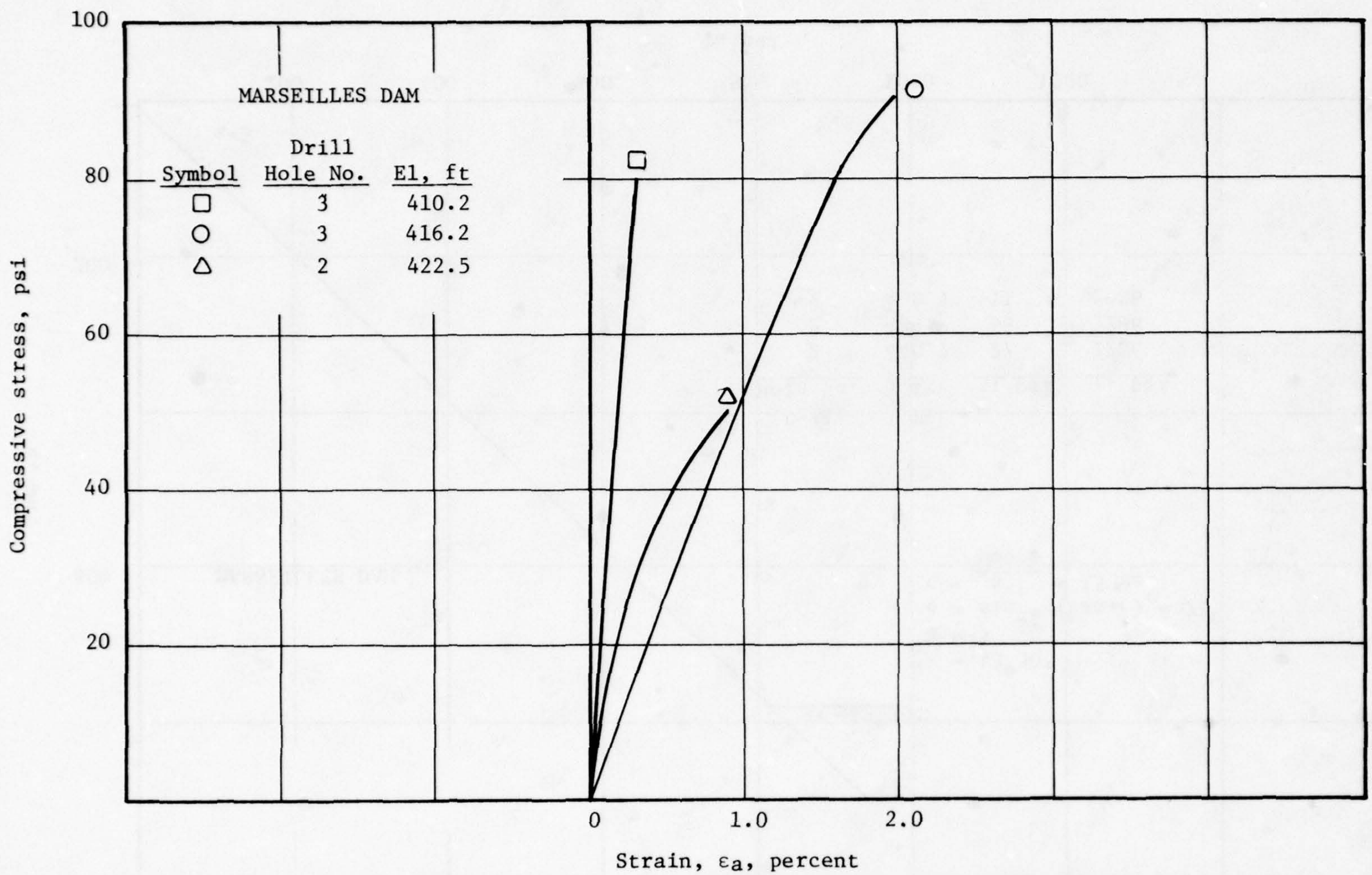
Composite plot of stess-strain relation for Zone 1 shale



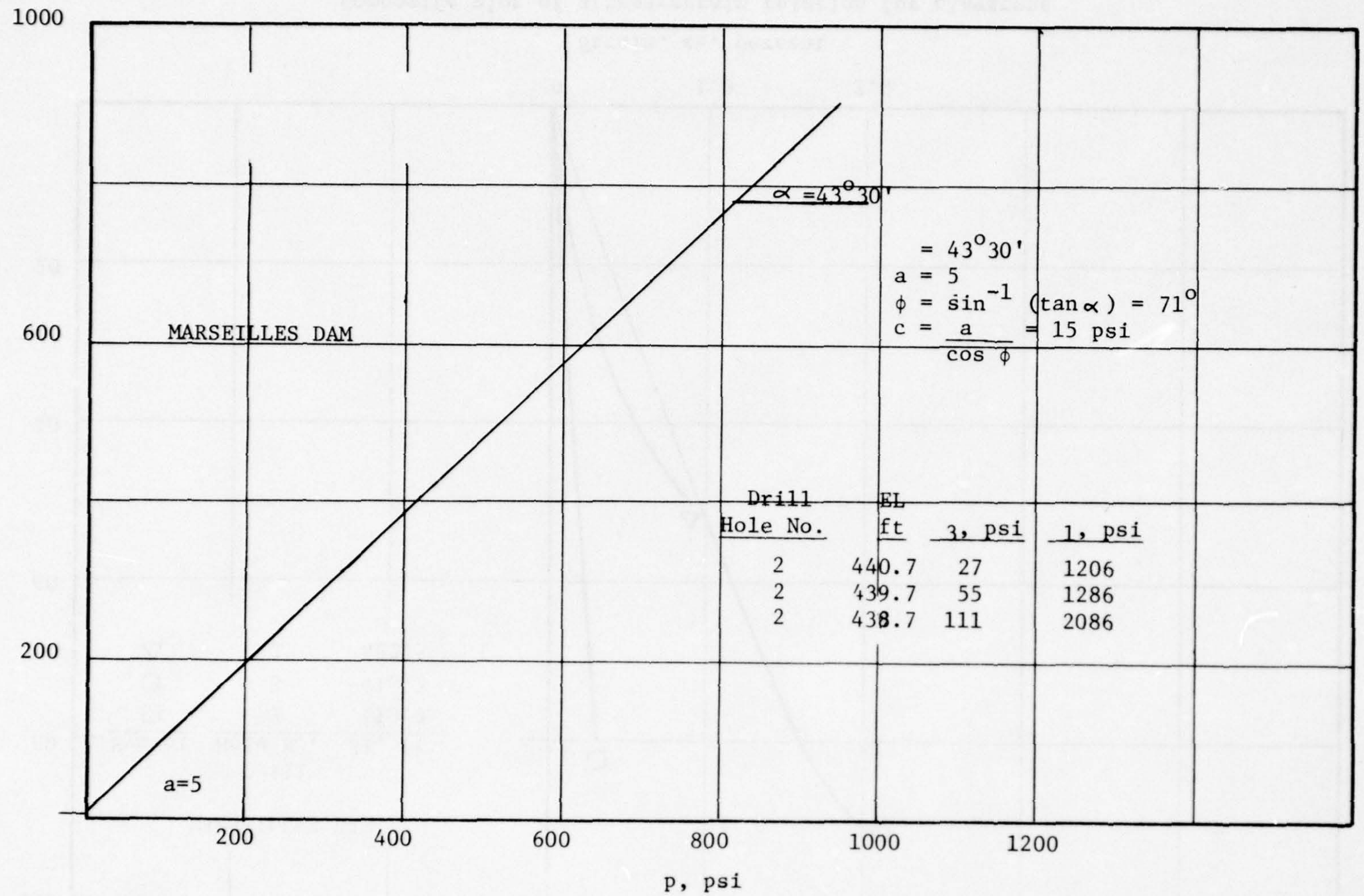
Composite plot of stress-strain relation for Zone 2 shale



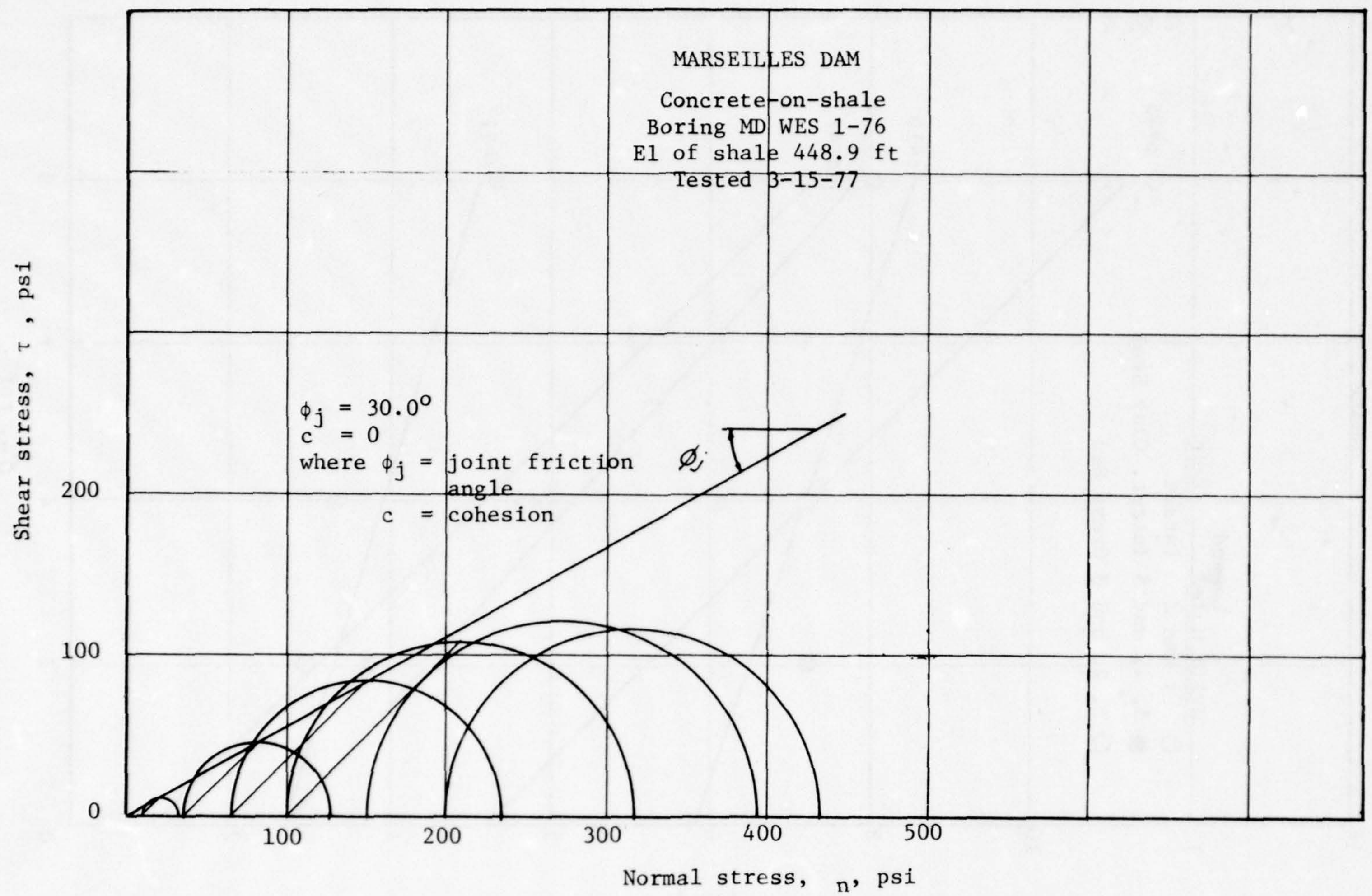
Stress-strain relation for coal



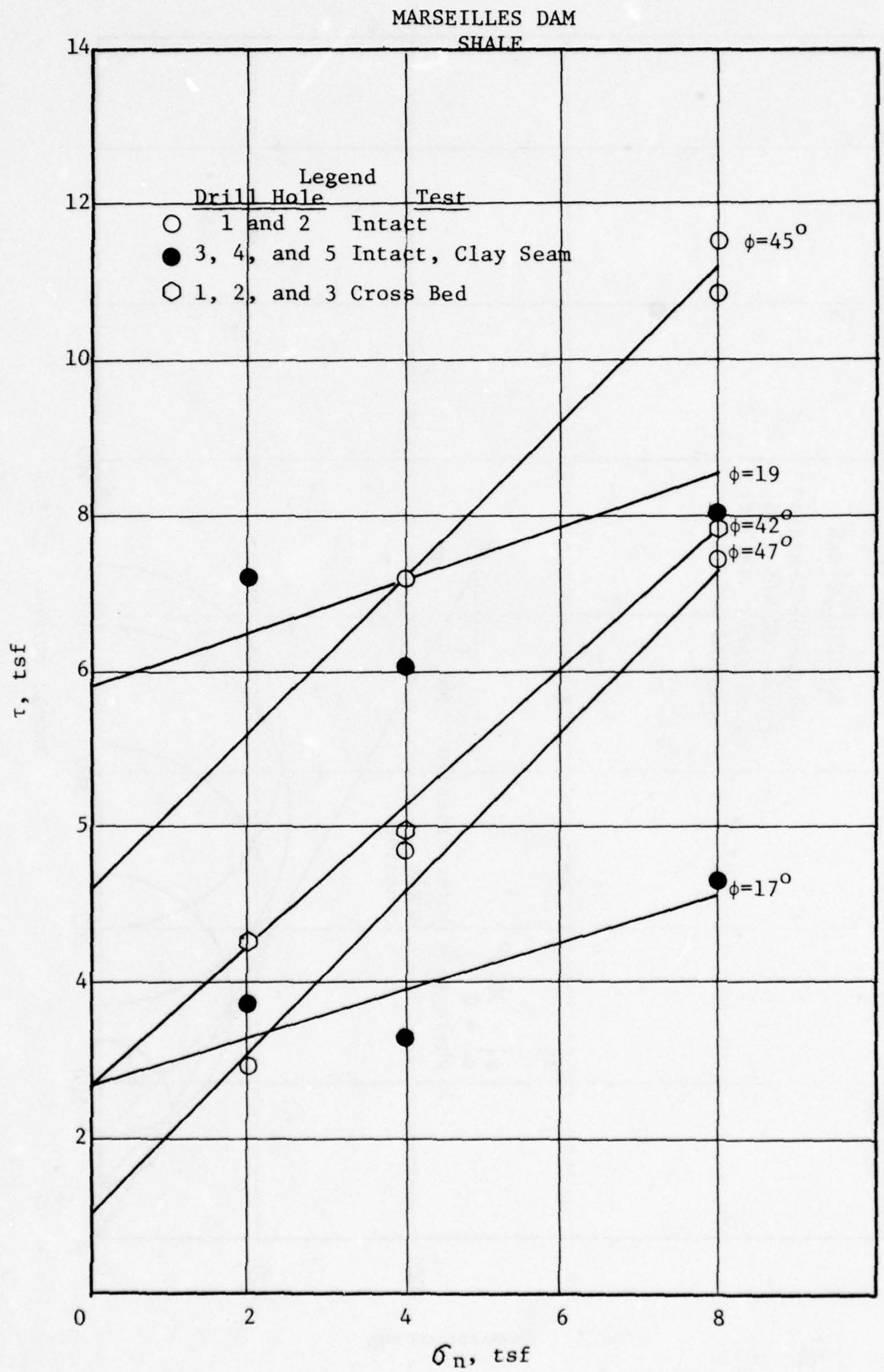
Composite plot of stress-strain relation for claystone



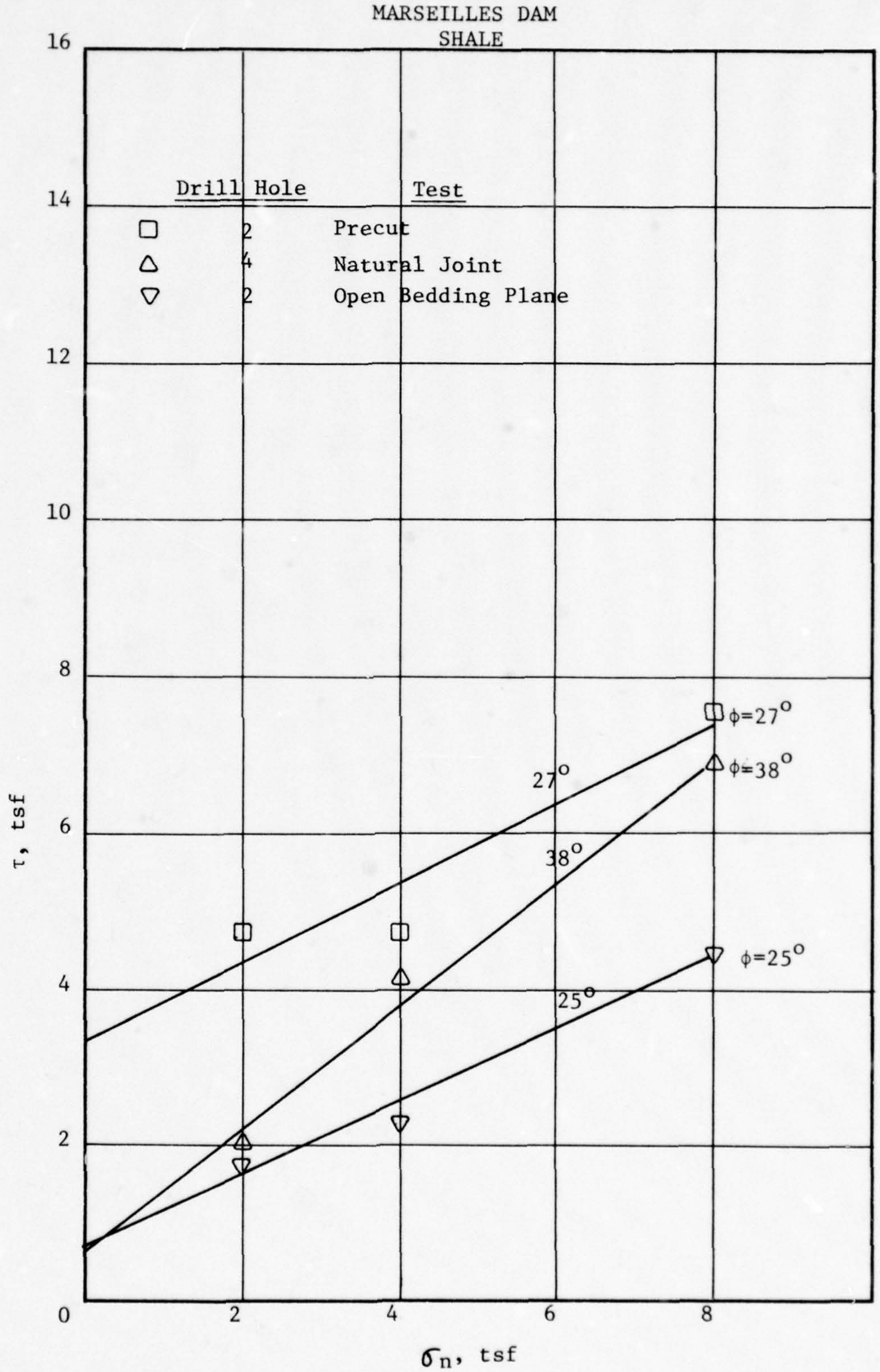
Results of triaxial tests on shale.



Mohr diagram for locating stresses on failure plane; multistage results.



Failure envelope for shale tested in direct shear



Failure envelopes for shale tested in direct shear

PLATE 13

APPENDIX A

PETROGRAPHIC REPORT

MARSEILLES DAM
ILLINOIS WATERWAY

Corps of Engineers, USAE Waterways Experiment Station	Petrographic Report	Concrete Laboratory P. O. Box 631 Vicksburg, Mississippi
Project Tests of Concrete and Rock Cores, Marseilles Dam, Illinois Waterway		Date 2 Jun 1977 GSW

Background

1. The U.S. Army Engineer District, Chicago, requested that the foundation rock and concrete at Marseilles Dam on the Illinois Waterway be evaluated as part of the stabilization of the structure. Drilling to obtain material for evaluation began in mid December 1976 and was temporarily discontinued early in February 1977 because of severe weather conditions and inability to obtain support to set up the drill at hole No. 5 which is located just downstream of the tainter gate pier. Hole No. 5 was drilled in May 1977.

2. The cores were logged in the field by three different inspectors. All of the cores were examined in more detail at WES using the field logs as needed. The inspectors also made a photographic log and selected lengths of concrete for further examination and testing. The rock and concrete cores selected for evaluation were immediately sealed in wax to retain field moisture, placed in insulated core boxes, and during subfreezing temperatures they were stored in a heated building until they were shipped to WES.

Samples

3. Five nominal 6-in.-diameter vertical cores were drilled at Marseilles Dam. The borings were identified in the field as MD WES-(1-5). Boring No. 1 is located in the nose of the pier between gate 7 and the ice chute. Gates are numbered starting from the Marseilles channel side and working towards the control house. Boring No. 2 is located in the nose of the pier between gates 4 and 5. Boring No. 3 is located on the west abutment of gate 1 (upstream portion), in line with boring No. 1 and 2. Borings 4 and 5 lie on a line perpendicular to borings 1, 2, and 3. Boring 4 is located in the upper pool in line with boring 2 and No. 5 is located in the lower pool in line with boring 2. The cores are identified and have been assigned Concrete Laboratory (CL) serial numbers as shown below:

CL Serial No. CHI-11	Boring No.	Depth, ft	Location	Description
CON-1 A	MD WES-1-76	0.0-1.0	Nose of pier between gate 7 and ice chute	6-in. vertical concrete core
B		1.5-4.5		
C		11.2-15.7		
D		20.7-25.0		
E		32.0-32.9		
DC-1		32.9-88.2		6-in. vertical rock core
CON-2 A	MD WES-2-77	0.0-4.6	Nose of pier between gates 4 and 5	6-in. vertical concrete core
B		27.9-33.6		

CL Serial No. CHI-11	Boring No.	Depth, ft	Location	Description
DC-2		33.6-88.4		6-in. vertical rock core
CON-3 A	MD WES-3-76	0.0-4.5	Left dam abutment	6-in. vertical concrete core
CON-3 B	MD WES-3-76	13.1-17.4 30.0-34.5	Left dam abutment	6-in. vertical concrete core
DC-3		34.5-85.6		6-in. vertical rock core
DC-4	MD WES-4-77	24.1-86.9	Upper pool in hole 2	6-in. vertical rock core
CON-4	MD WES-5-77	0.0-1.65	Lower pool in hole 2	6-in. vertical concrete core
DC-5		1.65-70.0		6-in. vertical rock core

4. Upon examination in the laboratory, a few of the rock cores were beginning to split on shale bedding and looked like poker chips in the core boxes. These pieces of core had dried as a result of moisture loss through damaged areas in the the wax coating. A few other pieces of core split into thin beds but were moist and did not show indications of loss of moisture. The cause of the splitting on bedding planes in some cases may have been drying, but the majority of splitting probably was caused by stress relief incurred in removing the rock from compaction caused by the burden of overlying rock or concrete.

Test procedures

5. Lengths of concrete core representing the different borings and different physical conditions of the concrete were selected by the inspector in the field for tests in the laboratory. The pieces of concrete selected near the surface were sawed parallel to the long axis of the core to allow better examination of the effects of frost action and possible deleterious reactions that may have also contributed to damaging the concrete.

6. Examinations of the rock were made to aid in determining the structural and physical characteristics of the various types of foundation rock. A detailed examination was made and all breaks, fractures, joints, contacts, change in rock type, and condition of the rock were logged. A visual comparison was made between the rocks at various depths within each boring and between each boring.

7. Pieces of rock cores representing the different rock types from the borings were examined with a stereomicroscope. A sample of each rock type was ground to pass a 45-μm sieve (No. 325) and then examined by X-ray diffraction since most of the mineral constituents were too fine to be identified using a polarizing microscope.

8. Samples of softer and more plastic material, considered to be clay, were also selectively removed from the rock core, air dried, and examined by X-ray diffraction as powders and as sedimented slides. These X-ray diffraction patterns were compared with patterns of the shale.

9. The X-ray diffraction examinations were made with an X-ray diffractometer using nickel-filtered copper radiation.

Results

10. Most of the concrete examined was in good condition. No problems were encountered during the drilling and extraction of the concrete cores from the structure. The concrete cores remained intact and only occasional breaks were observed. However, none of the breaks except those near the top of the structure were considered to be old breaks. Some of the breaks were associated with steel reinforcing bars that may have lowered the tensile strength when the core was pulled. The locations of these bars are shown in Plate 1 of the main report.

11. The concrete consisted of 1-1/2-in. maximum size natural gravel composed of carbonate and igneous coarse aggregate particles. The fine aggregate was a natural siliceous sand. No evidence of alkali-aggregate reactions was observed.

12. The concrete was well consolidated and did not show signs of segregation. This structure was built before the use of air entrainment in concrete. There were some small entrapped air voids found throughout the concrete. Deterioration of concrete was observed in cores from borings 1 and 2, limited to the near-surface concrete to depths of 1.5 ft and 1.8 ft, respectively. Concrete from boring 5 is underwater during subfreezing conditions and therefore has limited exposure to freezing and thawing. Boring 3 is located close to an abutment and is partially sheltered from the elements.

13. Most of the foundation rock was shale with a high content of clay minerals. If the rock was fissile it was called clay shale and if it lacked fissility it was called claystone.[2,3]

14. The rock in each boring usually consisted of a descending sequence of medium dark gray (N 4)[1] clay shale, coal, dark gray (N 3)[1] to brownish black (5YR 2/1)[1] claystone, dusky yellow green (5GY 5/2)[1] claystone with a medium light gray (N 6)[1] limestone bed, and finally very light gray (N 8)[1] sandstone. The limestone was detected only in two of the cores (Plate A1, MD WES-1, and Plate A2, MD WES-5). A chert conglomerate with a limestone matrix was present in core MD WES-4 (Plate A2) at about the same elevation that sometimes showed limestone. As indicated above, there was a considerable range of color in the claystone; it was all considered to be claystone regardless of color.

15. The different rock types are described below:

a. The foundation rock above the coal was a medium dark gray (N 4)[1] clay shale. All of the rock examined from this interval was composed of quartz, plagioclase feldspar, and siderite as the nonclay constituents.

The clay minerals present in this rock were clay-mica, kaolinite, and chlorite. The shale contained some nodules ranging from 10 mm to 80 mm in maximum dimension consisting of quartz, siderite, and pyrite. The shale was speckled with small pyrite crystals. There were indications of seams less than 2 mm thick of less indurated clay. The compositions of these seams was like that of the more indurated adjacent shale. The locations of these seams are indicated in the geologic cross sections (Plates A1,A2). The rock was competent, intact, and slightly fissile. However, the rock had a tendency to split into thin beds as described earlier in the report.

b. The coal was used as the marker bed for the geologic cross sections (Plates A1,A2). It was located at about elevation 424 to 429 ft and was about 2.5 ft thick. It was in large part highly fractured in the core as if the volume had been reduced since the coal was deposited. The fractures in some cases were entirely or partially filled with calcite. The fractures tended to be at 90 deg to each other and produced blocky particles of coal.

c. Dark gray to brownish black (5YR 2/1)[1] claystone about 5 ft thick is found directly below the coal. This rock was not fissile and is less indurated than the dark gray (N 3)[1] shale above the coal. The major constituents were quartz and kaolinite with minor amounts of plagioclase feldspar, pyrite, siderite, clay-mica, and possibly some mixed-layer clay and halloysite with two molecules of water. The rock is highly fractured and contains at least three sets of joints with slickensided surfaces produced by possibly contemporaneous deformation. The dip angles and assumed azimuths of these partings are indicated in structural cross sections A-A and B-B (Plates A3,A4).

d. Dusky yellow-green (5GY 5/2)[1] claystone is a bed about 1 ft thick located at about elevation 418 ft. The rock was composed of quartz, plagioclase feldspars, clay-mica, and mixed-layer clay-mica and smectite.* All borings contained this rock type except boring 5 (Plate A2). Rock in the core from boring 5 at about this elevation (418.5 ft) was a greenish gray (10Y 5/2)[1] rock, but this rock was composed of quartz, pyrite, kaolinite, clay-mica, and mixed-layer smectite and clay-mica. No kaolinite or pyrite was detected in the rock from other borings at the same elevation. The physical condition of this rock is like the dark gray (N 3)[1] claystone above.

e. The sequence of rock below the green claystone and above the very light gray (N 8)[1] sandstone is less continuous between the borings than the sequence of rock above it. The rock below the green claystone is mostly a gray claystone with the color varying from olive gray (5Y 4/1)[1] to dark gray (N 3)[1] with some organic debris in some of the dark gray (N 3)[1] rock. In boring 4 at about elevation 405 (Plate A2), there was a bed of green claystone about 1 ft thick. This rock consisted of quartz, pyrite, siderite, kaolinite, clay-mica, chlorite, and mixed-layer clay-mica, and smectite.

---

* Swelling clays of the montmorillonite-saponite group.

The gray claystone was generally composed of quartz, kaolinite, clay-mica, mixed-layer clay-mica, and smectite, with some stone nodules and pyrite crystals disseminated throughout. Borings 1 and 5 (Plates A1,A2) contained a limestone bed about 1 ft thick at elevations 415 ft and 412 ft, respectively. Boring 4 (Plate A2) had a bed of conglomerate about 1 ft thick made up of chert nodules less than 3 in. in diameter in a limestone matrix. The physical character of this sequence of rock ranged from very hard and competent limestone to soft, highly fractured claystone.

f. The last rock type encountered was a very friable, medium-grained, loosely cemented, very light gray (N 8)[1] sandstone. The top foot of sandstone was normally a gradation from clayey sandstone to sandstone. In a few locations this zone was cemented with iron oxide and was extremely hard. The sandstone became very friable as the depth increased.

Summary and discussion

16. The concrete in the cores is in good condition except for near-surface deterioration caused by freezing and thawing. Concrete deterioration caused by freezing and thawing occurred in the concrete of borings 1 and 2.

17. The foundation rock is essentially flat lying with a gentle dip from boring 1 toward boring 3, i.e., to the southeast. There were no indications of other major structural features.

18. The foundation rock above the coal bed had structural features typical of shale. There were many breaks along horizontal bedding planes with occasional thin clay seams up to 2 mm thick.

19. The coal was highly fractured. The rock between the coal and the sandstone consisted of highly fractured, slickensided, and occasionally very soft claystone. The limestone in borings 1 and 5 and the conglomerate in boring 4 were hard and intact. Because the inclined jointing in the rock between the coal and sandstone did not extend into the coal nor the sandstone, this evidence of activity was considered to be contemporaneous deformation. This is generally small scale deformation, faulting, jointing, that occurs between undisturbed beds.

20. The mineralogical constituents of the various rock types are given in Table A1. All of the rock types with the exception of the coal, limestone, and sandstone consisted mainly of quartz and clays. The shale above the coal and the green claystone at elevation 401-405 ft were the only rock types to contain chlorite. The interval of dark gray[1] claystone, green claystone, and gray claystone contained mixed-layer clay-mica and smectite, which was not found in the dark gray (N 3)[1] shale above the coal. Halloysite with two molecules of water was detected in the gray claystone of boring 1 and is believed to be also present in the gray claystone at approximately the same elevation in the other borings.

REFERENCES

1. The Rock-Color Chart Committee, National Research Council, Rock-Color Chart, Washington, D. C., 1963.

2. Dunbar, C. O., and Rogers, J., Principles of Stratigraphy, p 166, John Wiley and Sons, Inc., New York, 1957.

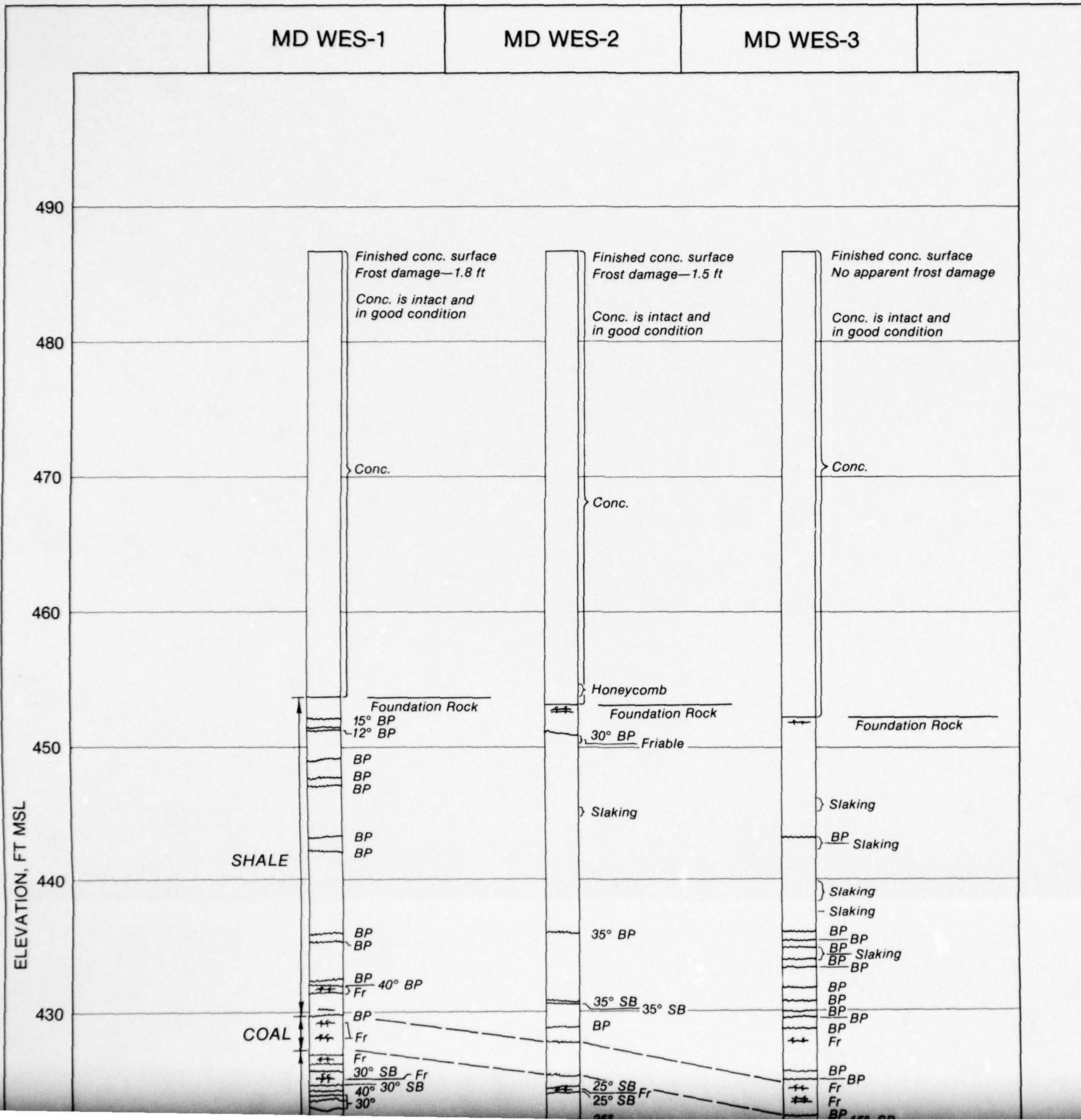
3. Dictionary of Geological Terms, pp 87-88, American Geological Institute, Doubleday and Co., Inc., New York, 1962.

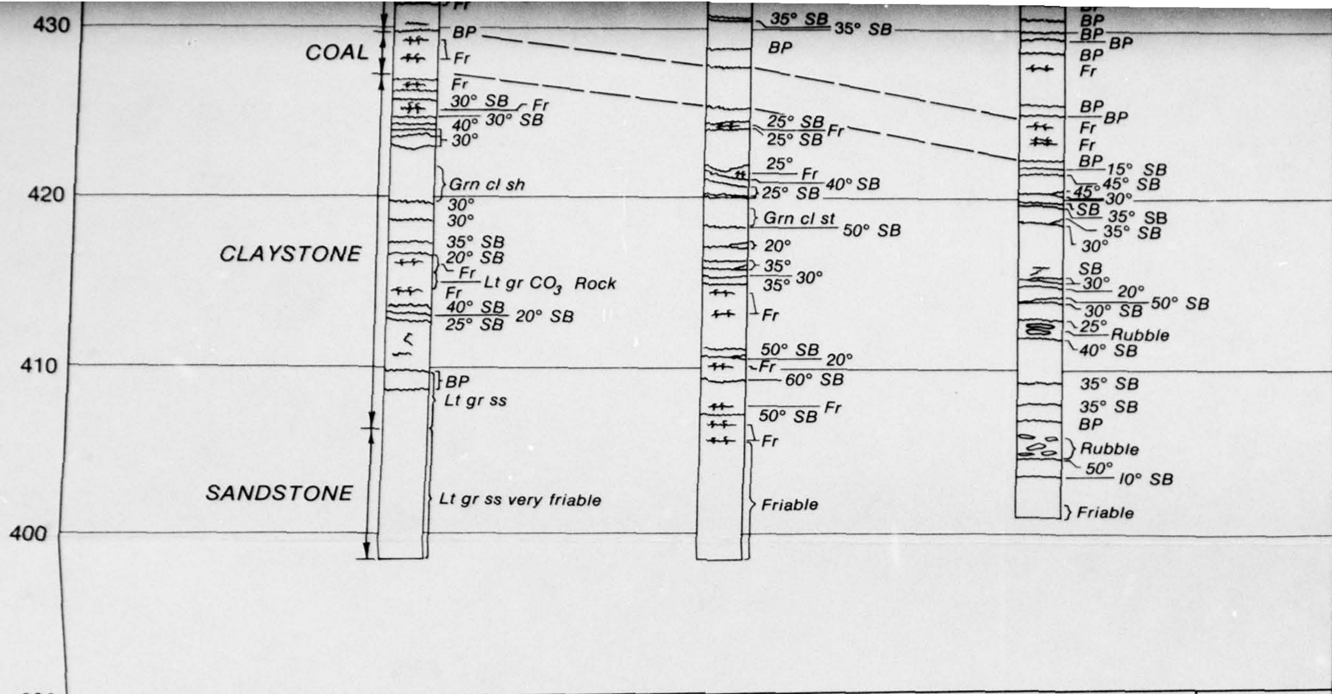
Table A1

Composition of Foundation Rock from Marseilles Dam, Holes WES-MD 1 through 5, by X-Ray Diffraction

Rock Type and Approximate El, ft msl	Nonclays					Clays			
	Quartz	Plagioclase Feldspar	Pyrite	Siderite	Calcite	Clay-Mica	Kaolinite	Chlorite	Mixed-Layer
Dark gray clay shale Top - 428	X	X	X	X		X	X	X	
Coal* 428 - 425					X				
Dark gray claystone 425 - 419	X	X	X	X		X	X		X
Green clay-stone 419 - 418	X	X	X			X	**		X
Gray clay-stone † 418 - 405	X					X	X		X
Limestone 411 - 412 ††	X				X	X	X		
Green clay-stone 406 - 405 ††	X		X	X		X	X	X	X
Sandstone ‡ 405 - bottom	X								

* Detected calcite by application of diluted hydrochloric acid.
** Kaolinite was detected only in boring 5.
† Halloysite with two water molecules was indicated in boring 1 by resolution into two peaks at about 7 and 3.5 Å.
†† Rock type not continuous.
‡ Analysis made only by visual examination.


MD WES-1
MD WES-2
MD WES-3
ELEVATION, FT MSL
490
480
470
460
450
440
430
Finished conc. surface
Frost damage—1.8 ft
Conc. is intact and
in good condition
Conc.
Foundation Rock
15° BP
12° BP
BP
40° BP
Fr
30° SB
40° 30° SB
30°
SHALE
COAL
Finished conc. surface
Frost damage—1.5 ft
Honeycomb
30° BP
Friable
Slaking
35° BP
35° SB
25° SB
Finished conc. surface
No apparent frost damage



LEGEND

- PREEXISTING PARTING
  BEDDING PLANE (BP)
  STRUCTURAL BREAK (SB)
- ANGLE OF DIP (25°)
- FRACTURED (Fr)

PREPARED FOR
U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
CHICAGO, ILLINOIS

MARSEILLES DAM
ILLINOIS WATERWAY, ILL.

BEDROCK STRUCTURAL CHARACTERISTICS
RELEVANT TO FOUNDATIONS

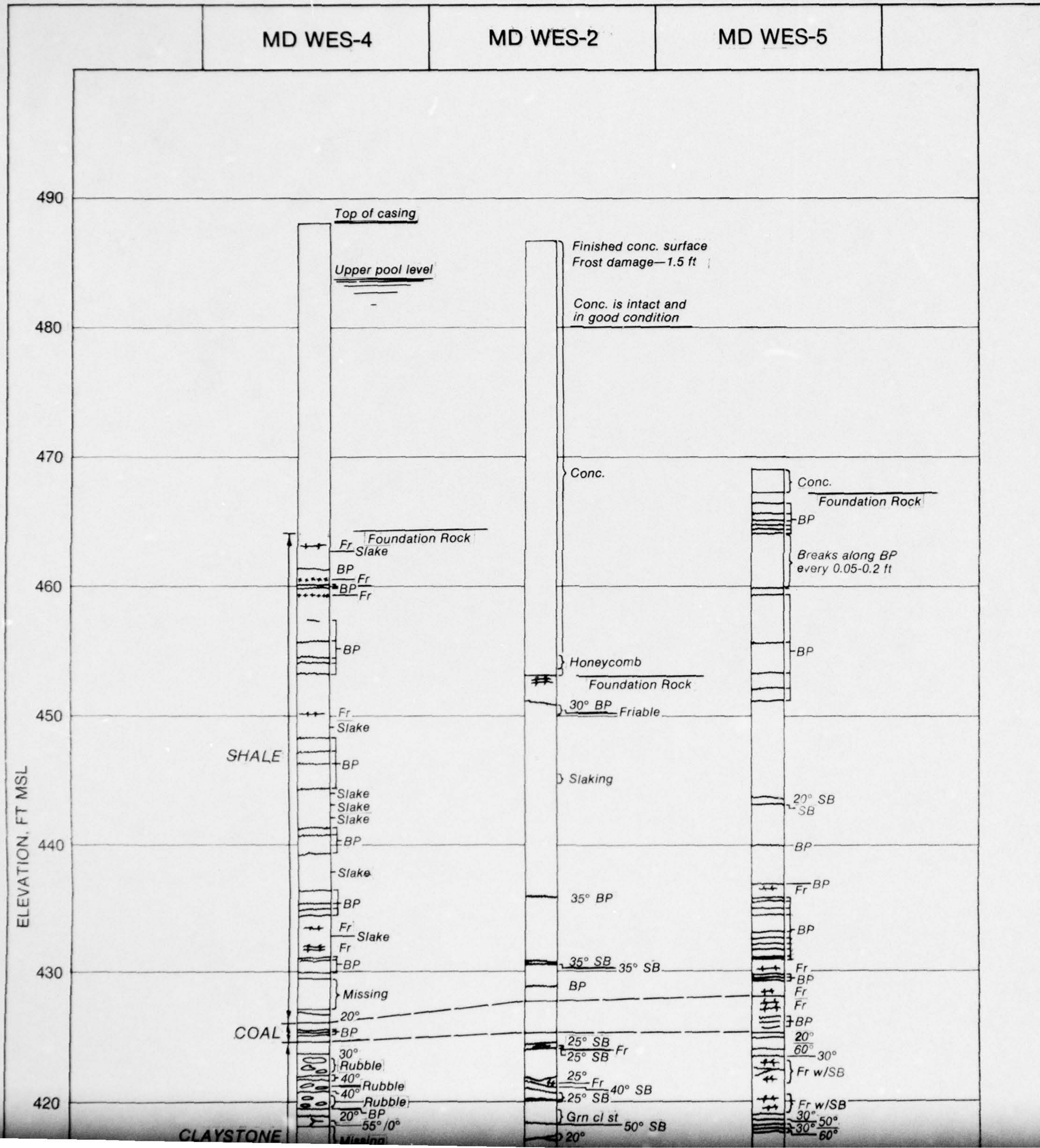
PREPARED BY ______
DRAWN BY ______
DATE ______
SHEET ___ OF ______

PREPARED BY
U. S. ARMY ENGINEER
WATERWAYS EXPERIMENT STATION
CORPS OF ENGINEERS
VICKSBURG, MISSISSIPPI

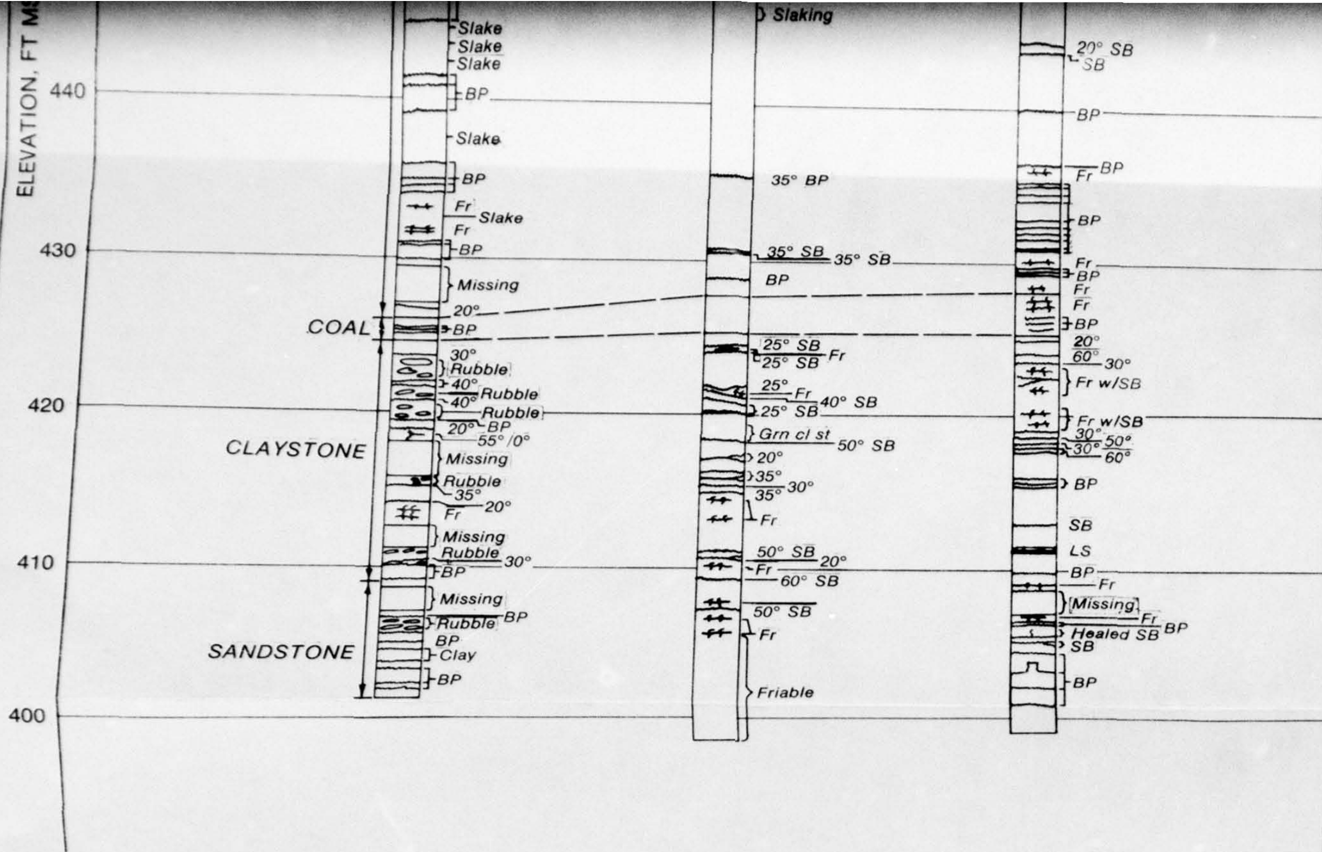
BEDROCK STRUCTURAL CHARACTERISTICS, SECTION A-A

PLATE A1

2



MD WES-4
MD WES-2
MD WES-5
ELEVATION, FT MSL
490
480
470
460
450
440
430
420
Top of casing
Upper pool level
Foundation Rock
Fr
Slake
BP
Fr
BP
Fr
BP
Fr
Slake
SHALE
BP
Slake
Slake
Slake
BP
Slake
BP
Fr
Slake
Fr
BP
Missing
20°
COAL
BP
30°
Rubble
40°
Rubble
40°
Rubble
20°
BP
55°/0°
CLAYSTONE
Missing
Finished conc. surface
Frost damage—1.5 ft
Conc. is intact and
in good condition
Conc.
Honeycomb
Foundation Rock
30° BP
Friable
Slaking
35° BP
35° SB
35° SB
BP
25° SB
25° SB
Fr
25°
Fr
40° SB
25° SB
Grn cl st
50° SB
20°
Conc.
Foundation Rock
BP
Breaks along BP
every 0.05-0.2 ft
BP
20° SB
SB
BP
BP
Fr
BP
Fr
BP
Fr
Fr
BP
20°
60°
30°
Fr w/SB
Fr w/SB
30°
50°
30°
60°



LEGEND

- PREEXISTING PARTING BEDDING PLANE (BP) STRUCTURAL BREAK (SB)
- ANGLE OF DIP (25°)
- FRACTURED (Fr)

PREPARED FOR
U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
CHICAGO, ILLINOIS

MARSEILLES DAM
ILLINOIS WATERWAY, ILL.

BEDROCK STRUCTURAL CHARACTERISTICS
RELEVANT TO FOUNDATIONS

PREPARED BY ______
DRAWN BY ______
DATE ______
SHEET ___ OF ______

PREPARED BY
U. S. ARMY ENGINEER
WATERWAYS EXPERIMENT STATION
CORPS OF ENGINEERS
VICKSBURG, MISSISSIPPI

BEDROCK STRUCTURAL CHARACTERISTICS, SECTION B-B'

PLATE A2

2

APPENDIX B

TELEVIEWER LOGS

MARSEILLES DAM

ILLINOIS WATERWAY

SEISVIEWER LOGS
CORPS OF ENGINEERS
MARSEILLES DAM PROJECT
LA SALLE CO., ILLINOIS

1. The SEISVIEWER records were taken on 6 May 1977. The records are labeled as to depth and orientation with the orientation scale referenced to magnetic north at the top of the log. See Plates B1-B4.

INTERPRETATION

2. The interpretation of the SEISVIEWER records is performed by a visual study, use of a scale and referring to the orientation shown at the top of the logged section.

3. Briefly stated, when a smooth normal surface is scanned by the transducer in the tool, the recorded image will consist simply of bright (white) traces from a good reflector. When a feature such as a fracture with its attendant discontinuities and irregularities is scanned, it presents a surface that is not normal to the scanning beam. The fracture then constitutes a poor reflector with the resultant black or dark image.

4. A vertical fracture is recognized by two nearly vertical lines. A fracture which is tilted from the horizontal and vertical is recognized by a sinusoidal curve with a single maximum and minimum. The orientation of the minimum is the direction of dip. The tangent of the angle of dip can be computed by measuring the maximum to minimum distance of the sinusoidal curve and dividing by the hole diameter from the caliper.

MD WES-1

DEPTH	FEATURE	REMARKS
5-34	Irregular orientation	Reinforcing bars in concrete disrupted magnetic orientation of tool.
27-29-1/2	Tool Malfunction	Dark areas are not valid due to tool malfunction.
48	Fracture	Angle of dip of fracture plane is 13° NE.
55-86	Hole enlargements	The dark areas are rough or irregular sections of hole primarily along bedding or fracture planes.

(Continued)

DEPTH	FEATURE	REMARKS
64	Fracture	Angle of dip of fracture plane is 20° SSE. Several indistinct fractures of about the same angle of dip can be seen approximately 5 ft above and below this fracture.
74-75	Fracture	Angle of dip of fracture plane is 34° SE.
78	Fracture	Angle of dip of fracture plane is 34° SE.

MD WES-2

0-34	Irregular orientation	Reinforcing bars in concrete disrupted magnetic orientation of tool.
32-34	Enlarged hole	Irregular hole.
50-51	Fracture	Angle of dip of fracture plane is 39° South.
57.5-58.5	Fracture	Angle of dip of fracture plane is 31° East.
50-84	Hole enlargements	Dark portions reflect rough or irregular hole.

MD WES-3

0-34.5	Irregular orientation	Reinforcing bars in concrete disrupted magnetic north orientation of tool.
13-14.5	Fracture	66° dip of fracture plane. Orientation uncertain due to reinforcing bars.
23.5	Fracture	Horizontal break or separation.
25	Fracture	Horizontal break or separation.
58.5-59.5	Hole Enlargement	Irregular hole.
62-83	Hole Enlargement	Irregular hole.
72-83	Vertical Fracture	Indistinct, due to irregular hole, intersects hole SE-NW.

MD WES-5

5-41	Key seat	Long Vertical feature where drill pipe wore a groove in borehole wall.

(Continued)

DEPTH	FEATURE	REMARKS
10-30	Vertical fracture	Indistinct over portions of interval, intersects hole NE-SW.
38-63	Hole enlargement	Dark portion reflect rough or irregular hole.

BIRDWELL

SEISVIEWER

COMPANY COPR. OF ENGINEERS

WELL MD WES-1

FIELD MARSEILLES DAM PROJECT

COUNTY LA SALLE STATE ILL.

LOCATION:	OTHER SERVICES
PIER NEXT TO ICE CHUTE SEC. TWP. RGE.	

ANENT DATUM TOP OF CMT. PIER , ELEV. 486.7 — ELEV. K.B. ____

MEASURED FROM TOP OF CMT. PIER Ft. Above Perm. Datum — D.F. ____

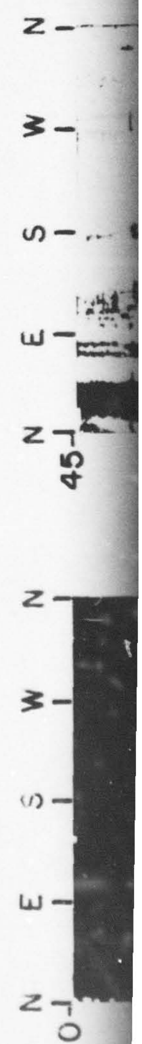
ING MEASURED FROM N.A. — G.L. 486.7

E	MAY 6, 1977			
NO.	1			
LOG	SVS			
H — DRILLER	89.2			
H — LOGGER	89			
LOG. INTER.	86			
LOG. INTER.	0			
FLUID IN HOLE	WATER			
LINITY PPM CL.				
NSITY LB./GAL.				
VEL	FULL			
.REC.TEMP.—°F				
. RIG TIME				
ORDED BY	SCHRODER			
IESSED BY	MR. B. NEAL			
ATION	11			

N	BORE HOLE RECORD			CASING RECORD			
.	BIT	FROM	TO	SIZE	WGT.	FROM	TO
	8	0	T.D.				

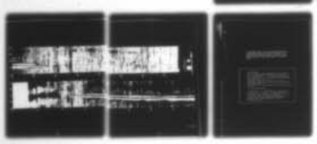
SSC-608-E

REMARKS:
NA: INFORMATION NOT AVAILABLE
ERRATIC TOOL RESPONSE IS DUE TO REINFORCING BARS IN CONCRETE.

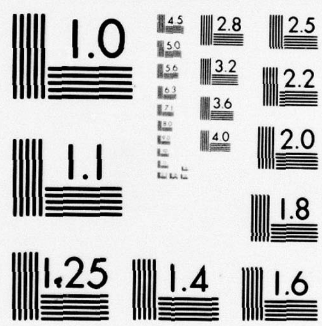


AD-A077 372 ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG--ETC F/G 13/13
CONCRETE AND ROCK TESTS, REHABILITATION WORK, MARSEILLES DAM, I--ETC(U)
SEP 79 R L STOWE
UNCLASSIFIED WES-SL-79-21 NL

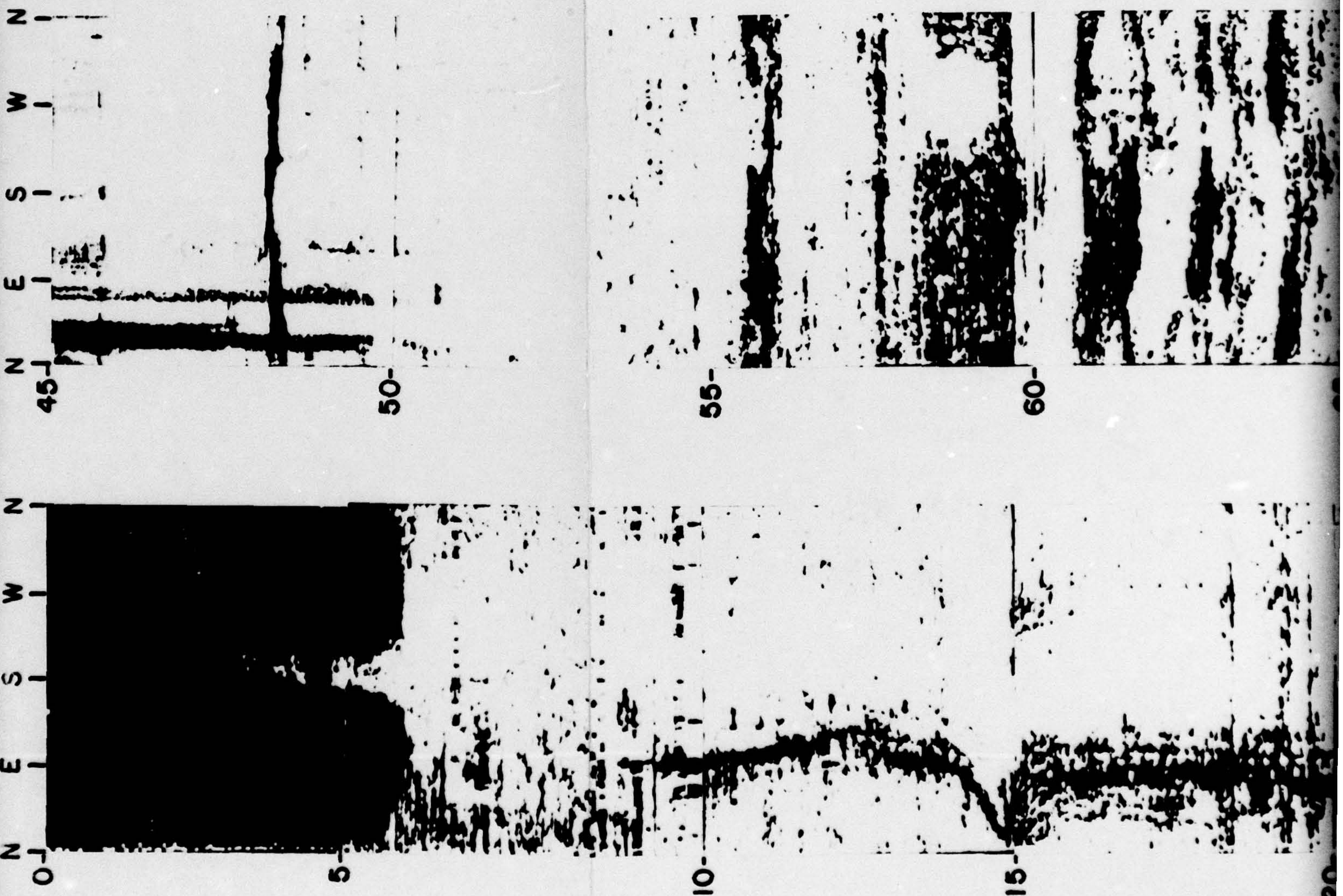
2 OF 2
ADA
077372



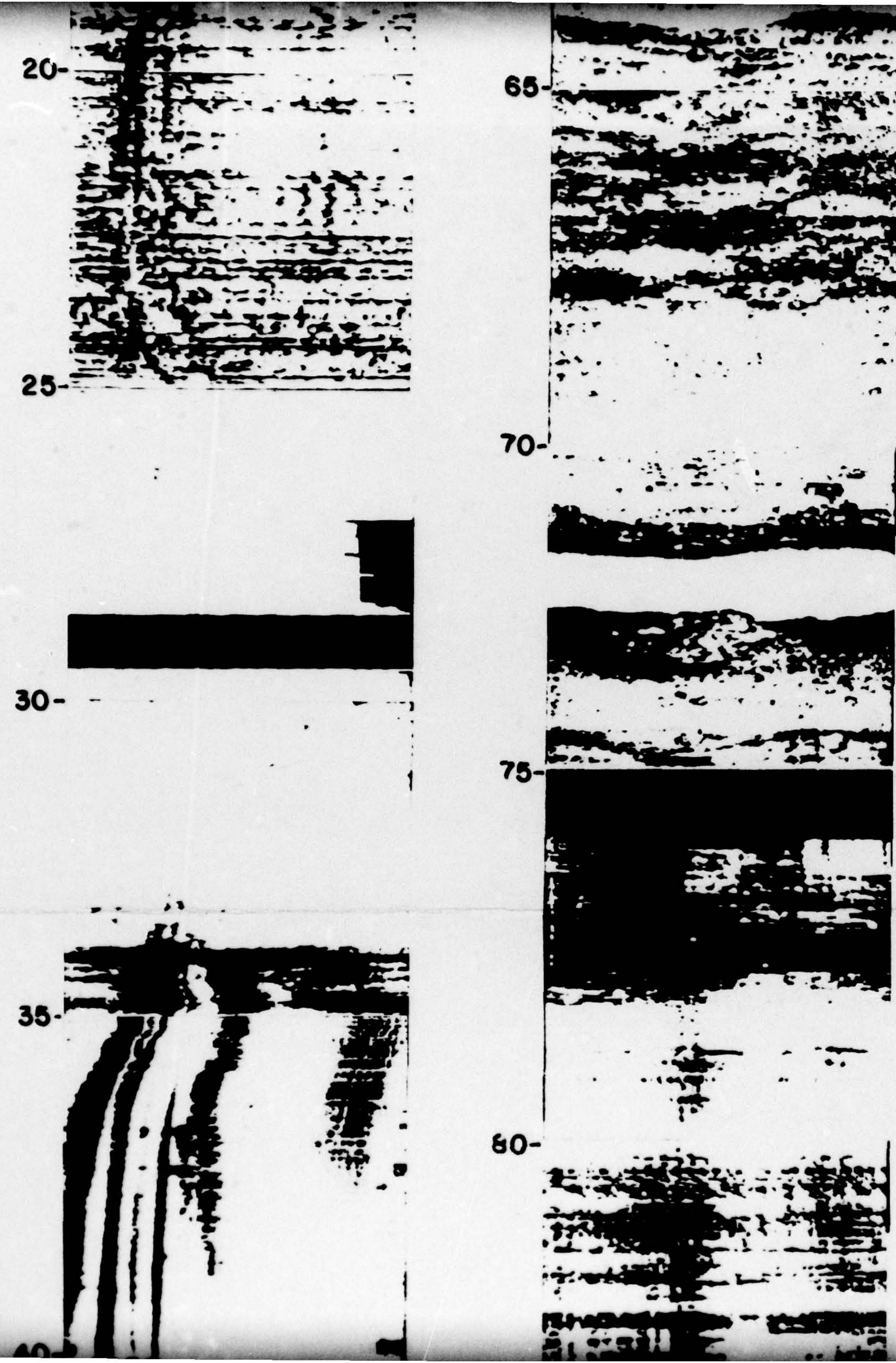
END
DATE
FILMED
12-79
DDC



MICROCOPY RESOLUTION TEST CHART



2



3

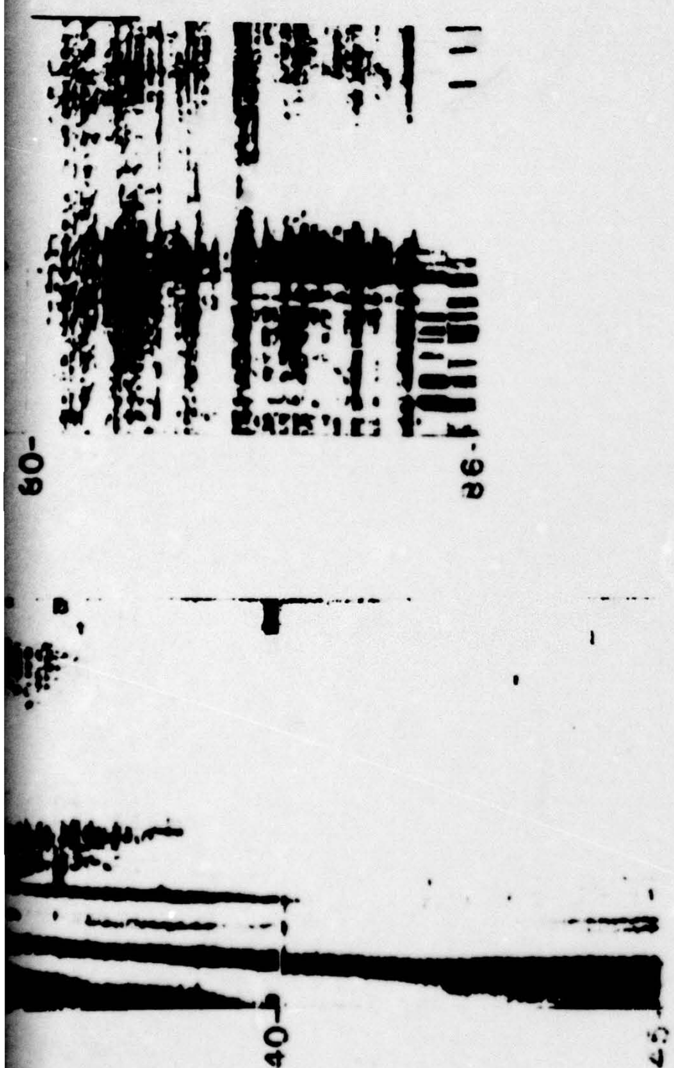


PLATE B1

4

1

BIRDWELL

SEISVIEWER

COMPANY COPR. OF ENGINEERS

WELL MD WES-1

FIELD MARSEILLES DAM PROJECT

COUNTY LA SALLE STATE ILL.

LOCATION:
PIER NEXT TO ICE CHUTE

SEC. TWP. RGE.

OTHER SERVICES

PERMANENT DATUM TOP OF CMT. PIER, ELEV. 486.7
LOG MEASURED FROM TOP OF CMT. PIER Ft. Above Perm. Datum
DRILLING MEASURED FROM N.A.

ELEV. K.B.
D.F.
G.L. 486.7

DATE	MAY 6, 1977			
RUN NO.	1			
TYPE LOG	SVS			
DEPTH – DRILLER	89.2			
DEPTH – LOGGER	89			
BTM. LOG. INTER.	86			
TOP LOG. INTER.	0			
TYPE FLUID IN HOLE	WATER			
SALINITY PPM CL.				
DENSITY LB./GAL.				
LEVEL	FULL			
MAX.REC.TEMP.–°F				
OPER. RIG TIME				
RECORDED BY	SCHRODER			
WITNESSED BY	MR. B. NEAL			
LOCATION	11			

RUN NO.	BORE HOLE RECORD			CASING RECORD			
	BIT	FROM	TO	SIZE	WGT.	FROM	TO
	8	0	T.D.				

SSC-608-E

REMARKS:
NA: INFORMATION NOT AVAILABLE
ERRATIC TOOL RESPONSE IS DUE TO REINFORCING BARS IN CONCRETE.

N W S E N N W S E N


N
W
S
E
N
45
50
55
60
N
W
S
E
N
0
5
10
15


65
70
75
60
20
25
30
35

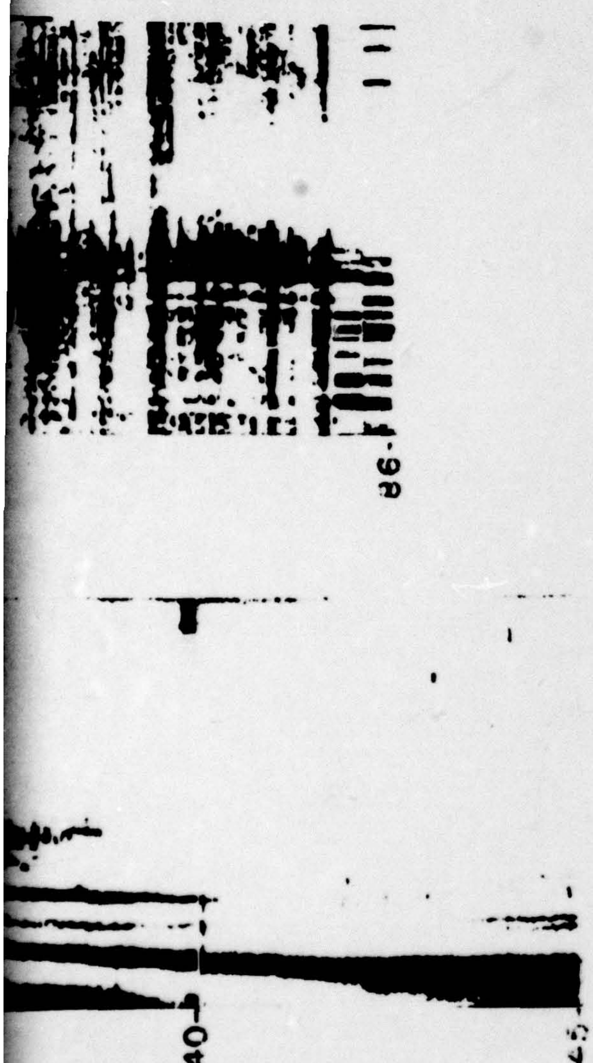


PLATE B1

1 4

BIRDWELL

SEISVIEWER

COMPANY CORP OF ENGINEERS

WELL MD WES-3

FIELD MARSEILLES DAM PROJECT

COUNTY LA SALLE STATE ILL.

LOCATION:

LEFT DAM ABUTTMENT

SEC. TWP. RGE.

OTHER SERVICES

PERMANENT DATUM TOP OF CMT. PIER , ELEV. 486.7

LOG MEASURED FROM TOP OF CMT. PIER Ft. Above Perm. Datum

DRILLING MEASURED FROM N.A.

ELEV. K.B.

D.F.

G.L. 486.7

DATE	MAY 6, 1977			
RUN NO.	1			
TYPE LOG	SVS			
DEPTH — DRILLER	85.6			
DEPTH — LOGGER	86			
BTM. LOG. INTER.	83			
TOP LOG. INTER.	0			
TYPE FLUID IN HOLE	WATER			
SALINITY PPM CL.				
DENSITY LB./GAL.				
LEVEL	FULL			
MAX.REC.TEMP.—°F				
OPER. RIG TIME				
RECORDED BY	SCHRODER			
WITNESSED BY	MR. B. NEAL			
LOCATION	11			

RUN NO.	BORE HOLE RECORD			CASING RECORD			
	BIT	FROM	TO	SIZE	WGT.	FROM	TO
	8	0	TD				

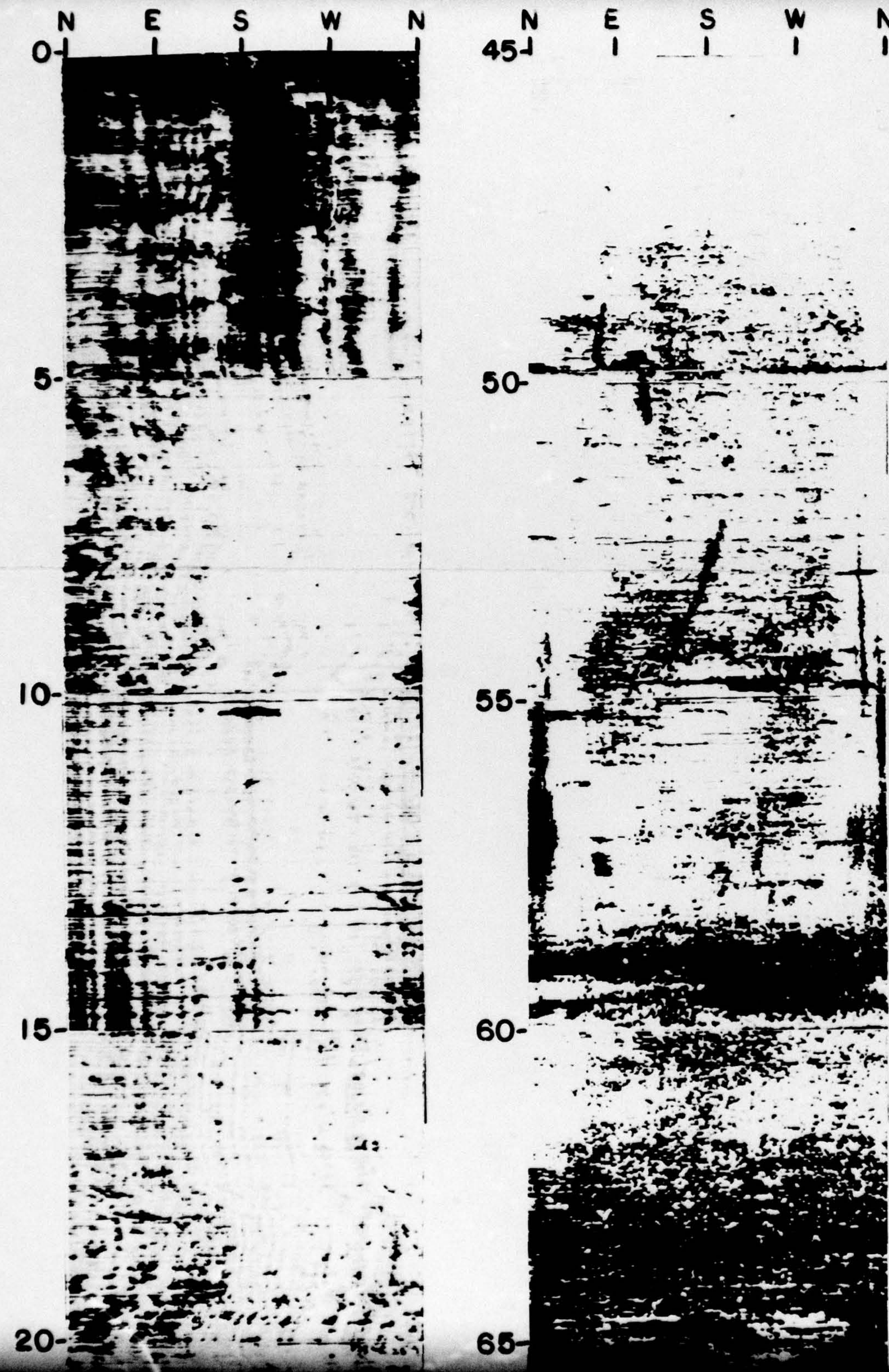
SSC-608-E

REMARKS:

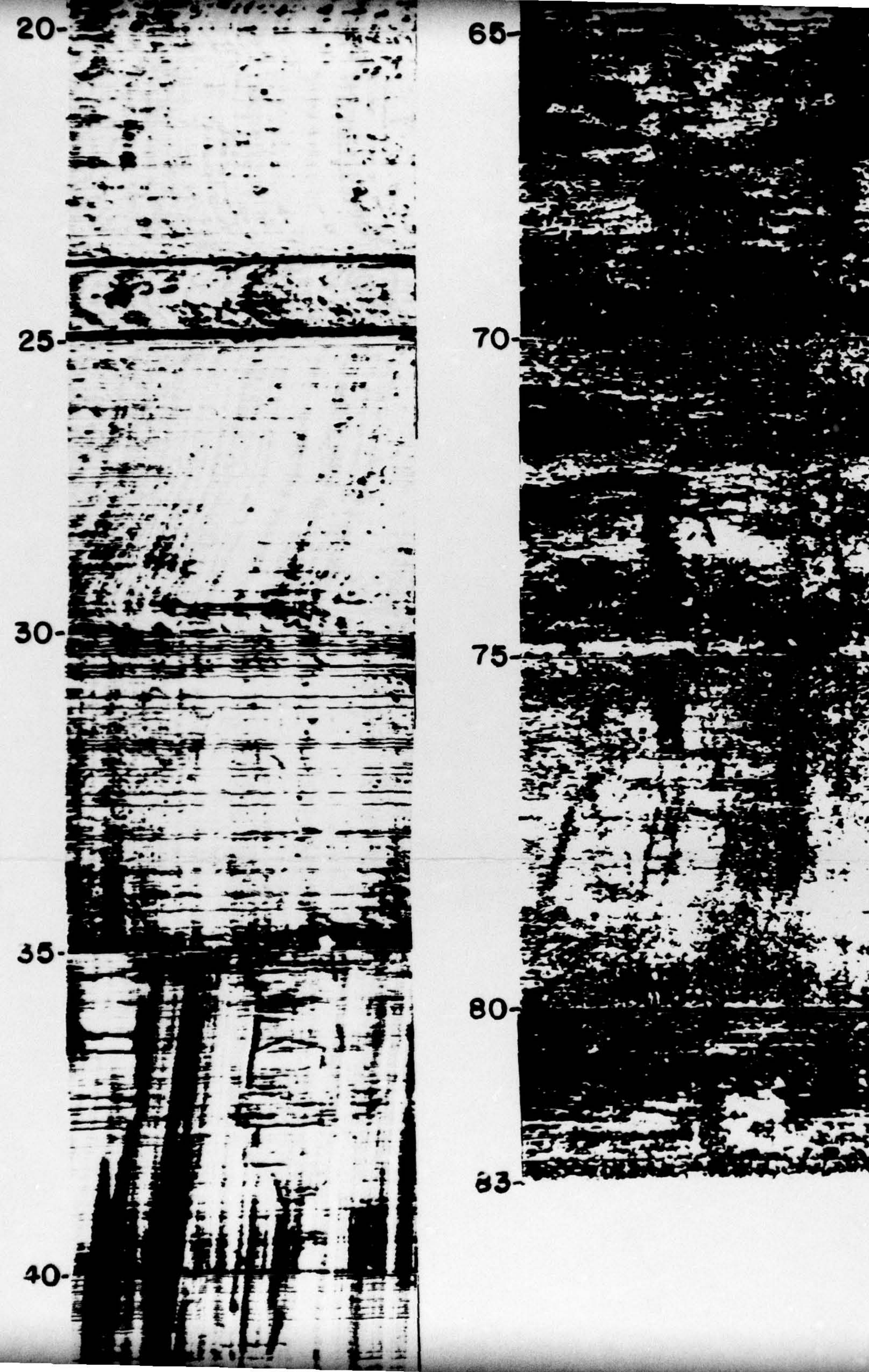
NA: INFORMATION NOT AVAILABLE

ERRATIC TOOL RESPONSE IS DUE TO REINFORCING BARS IN CONCRETE.

N W S E N 45 W S E N



2


20-
25-
30-
35-
40-
65-
70-
75-
80-
83-

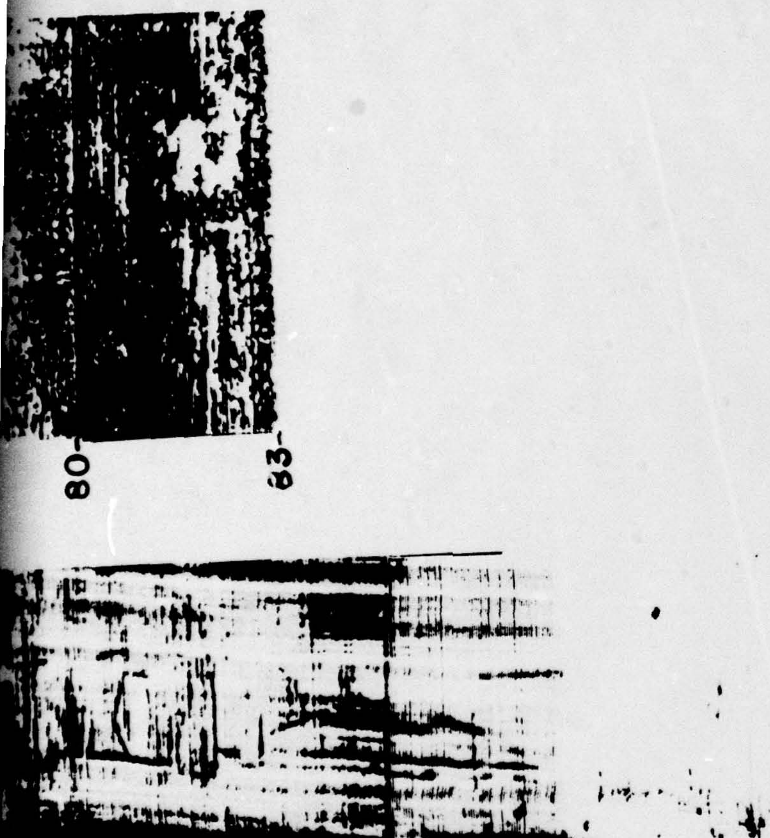


PLATE B3

4

BIRDWELL

SEISVIEWER

COMPANY CORP. OF ENGINEERS

WELL MD WES-5

FIELD MARSEILLES DAM PROJECT

COUNTY LA SALLE STATE ILL.

LOCATION: BEHIND DAM

SEC. TWP. RGE.

OTHER SERVICES

PERMANENT DATUM TOP OF CMT. LANDING, ELEV. N.A.

LOG MEASURED FROM TOP OF CMT. LANDING Ft. Above Perm. Datum

DRILLING MEASURED FROM N.A.

ELEV. K.B.

D.F.

G.L. N.A.

DATE	MAY 6, 1977			
RUN NO.	1			
TYPE LOG	SVS			
DEPTH — DRILLER	70			
DEPTH — LOGGER	69			
BTM. LOG. INTER.	66			
TOP LOG. INTER.	-5			
TYPE FLUID IN HOLE	WATER			
SALINITY PPM CL.				
DENSITY LB./GAL.				
LEVEL	FULL			
MAX.REC.TEMP.—°F				
OPER. RIG TIME				
RECORDED BY	SCHRODER			
WITNESSED BY	MR. B. NEAL			
LOCATION	11			

RUN NO.	BORE HOLE RECORD			CASING RECORD			
	BIT	FROM	TO	SIZE	WGT.	FROM	TO
	8	0	T.D.				

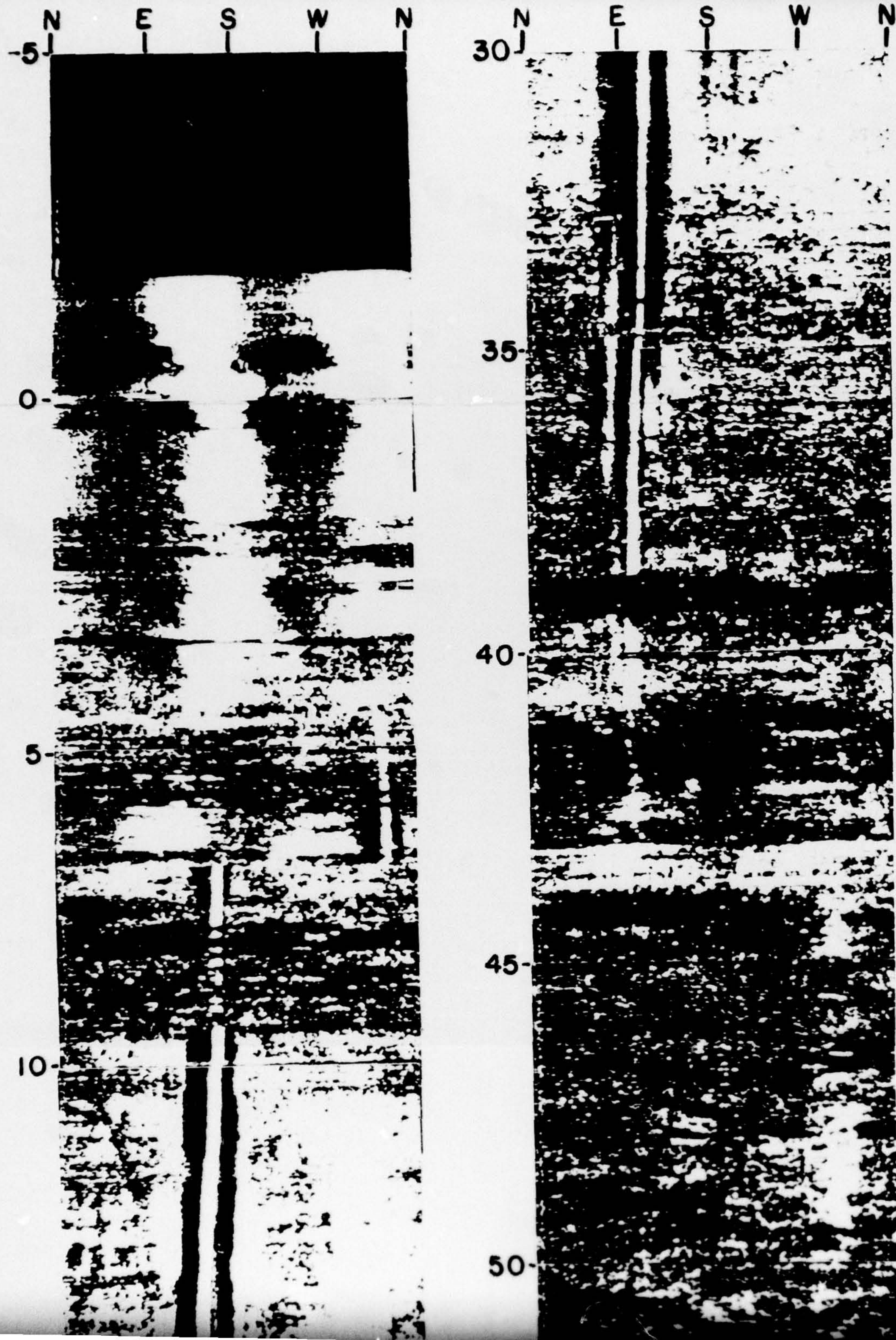
SSC-608-E

REMARKS:

NA: INFORMATION NOT AVAILABLE

ERRATIC TOOL RESPONSE IS DUE TO REINFORCING BARS IN CONCRETE.

N W S E N N W S E N



2

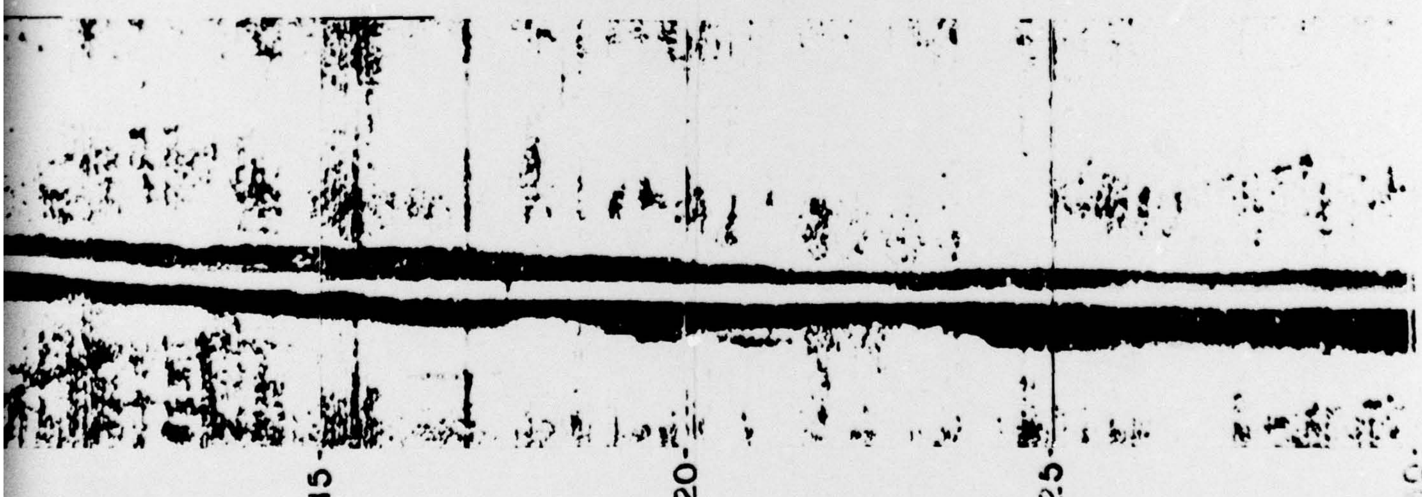


PLATE B4

3

In accordance with letter from DAEN-RDC, DAEN-ASI dated 22 July 1977, Subject: Facsimile Catalog Cards for Laboratory Technical Publications, a facsimile catalog card in Library of Congress MARC format is reproduced below.

Stowe, Richard L

Concrete and rock tests, rehabilitation work, Marseilles Dam, Illinois Waterway, Chicago District / by Richard L. Stowe. Vicksburg, Miss. : U. S. Waterways Experiment Station ; Springfield, Va. : available from National Technical Information Service, 1979.

38, [18] p., [19] leaves of plates : ill. ; 27 cm. (Miscellaneous paper - U. S. Army Engineer Waterways Experiment Station ; SL-79-21)

Prepared for U. S. Army Engineer District, Chicago, Chicago, Ill.

Includes bibliographies.

1. Concrete tests. 2. Dam foundations. 3. Dam stability. 4. Field tests. 5. Grouting. 6. Illinois Waterway. 7. Marseilles Dam. 8. Rock tests (Laboratory). I. United States. Army. Corps of Engineers. Chicago District. II. Series: United States. Waterways Experiment Station, Vicksburg, Miss. Miscellaneous paper ; SL-79-21

TA7.W34m no.SL-79-21